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Reinforced concrete sleepers,

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Figs. 1 to 44, pp. 115 to 166.

(*Rivista dei Trasporti.*)

Foreword. — It is generally known that for more than twenty years the principal Railway Companies throughout the world have been seriously concerned by the gradual exhaustion of the supplies of wood for railway sleepers. It is also known that their concern continually augments as a result of the continuous growth of railways, the increasing scarcity of suitable timber and the fantastic advances in price. It is these facts which have justified the various trials, namely, of metal and reinforced concrete sleepers, made to find a substitute for those in wood.

Another commonly known fact is that the problem is still more acute for European countries in general, and particularly for the Italian railways. The Italian forests largely cut into during the construction of the main net work of railway lines, and then required to contribute during the 50 years following towards extensions and the ever growing repairs, were again reduced, possibly excessively, and without regard to the future, by the war, so that to-day the available supplies have been considerably lessened and can only with the greatest difficulty meet the demands. The scarcity of good stock and of fully grown timber has resulted in a lowering of the quality of sleepers, and consequently in their length of service, whilst increasing the cost of maintenance.

The types of woods used for railway sleepers are English oak, white oak, beech and pine.

The first of these types can be utilised without preliminary creosoting, which is necessary with the others. A good English oak sleeper, even untreated, can safely be left twenty years or more in the track. Unfortunately, as we have already said, the English oak that is obtained nowadays is almost always drawn from young wood, with the result that the sleepers are almost entirely cut out of sap wood, which naturally reduces the length of service. For all the other kinds of wood, treatment with some form of preservative is absolutely necessary; a relatively longer service is thus obtained, but it is difficult to obtain a longer service than ten years.

The consumption of sleepers in Italy by the large State Railways has risen to-day to about 3 millions a year, and this number is certainly not less than 4 millions if the smaller lines and tramways are included. This enormous and ever increasing demand is still largely supplied by the forests of Italy, but the importation of wood is beginning to become interesting from an economical point of view, as the time is drawing near when it will become an absolute necessity.

Whilst other nations, richer in iron ore deposits, have partly solved the problem by adopting metal sleepers on a large scale, Italy is obliged to consider the adoption of concrete sleepers which to-day can alone give an economical solution, and avoid, more or less, her dependence on foreign industries.

It is well known that metal sleepers still present serious well known difficulties, such as weak transverse stability, too great deterioration in places where it is difficult to avoid stagnant water, costly maintenance beside which they easily get out of shape, etc.

It is therefore necessary that, without attaching too great importance to the lack of success so far experienced the problem of reinforced concrete sleepers be tackled with enthusiasm and keenness, whilst taking into consideration the long experience with these sleepers, until a satisfactory solution of the problem is arrived at.

The present notes are intended to give a short summary of the most important work on this matter to date, even if the results are far from encouraging; research into the probable causes of failures, together with brief information on the calculation of the dimensions and proportions of reinforced concrete sleepers; and lastly, a detailed account of a type of composite sleeper made of reinforced concrete and steel which is the most promising of all the types of sleepers proposed up to now, both because it fulfils the requirements of a well thought out design with rational dimensions, a point never overlooked even in our tentative proposals, and (what is most important) because of the excellent results obtained during lengthy tests.

PART I.

Examination of the principal types of sleepers.

Advantages and disadvantages of concrete sleepers. — Before proceeding to a detailed investigation into the reinforced concrete sleeper, it must be noted that all existing types, and probably all future types, will always be inferior to the wooden sleeper, from the point of view of the specific duty a railway sleeper has to fulfil namely, to connect the two rails and to transmit the pressures and shocks caused by a passing train to the ballast, and consequently to the formation.

The mechanical strength, elasticity, together with the resistance to atmospheric conditions, the small weight, the practical freedom from breaking, and the homogeneous composition of a good oak sleeper, will be very hard to obtain with any other system of sleeper whatever it may be.

Having first of all made these preliminary remarks, the chief characteristic drawbacks of reinforced concrete sleepers compared with wooden sleepers may be examined. It will be seen that certain of these defects can be corrected and that the other defects, which always result in the concrete sleeper being inferior to the ideal as represented by the wooden, are not likely however to be of such nature as to hinder its steadily extended use.

The first drawback of the concrete sleeper is its considerable weight. Whatever the type of concrete sleeper provided it is rationally designed, it can scarcely weigh less than 180 kgr. (397 lb.), or more than double the weight of a wooden sleeper. This increase in weight not only adds to the costs of transport, but also increases the cost of labour in laying the sleepers.

Another drawback to the concrete sleeper, which is not without its importance owing to the increase in weight, is its brittleness and relative weakness; owing to its very nature the concrete sleeper needs considerably more care in handling than the wooden sleeper.

The very relative value of these two defects should be considered straight away. As will be shown further on, the importance of the weight of concrete sleepers, especially in regard to carriage, is very small, seeing that these sleepers *ought never* to be carried great distances and that they can under most conditions be constructed at or near the spot where they are to be laid.

As to the increased cost of laying, the difference in the total cost of the sleeper in service will only be slight.

Lastly, if the brittleness of the concrete sleeper is greater, it is not sufficiently so to give rise to the least apprehension. The gangs of men engaged on the maintenance of the track will easily adapt themselves to the new method which necessitates greater care and a slower rate of working. The author has been present during the laying of thousands of these concrete sleepers, and has only on very rare occasions seen sleepers rejected for cracks or breaks, or defects due either to transport or the act of laying.

Against this, great care must be taken during the tamping of the sleepers, as a very light blow of the pick can partly break away the corners of concrete sleepers.

It has been found that flaking even often repeated can in no way affect the strength and durability of a concrete sleeper, and, further, will have no more serious consequences than in the case of wooden sleepers, in which careless blows from the pick split away the edges. This slight drawback, however, will ultimately disappear as the adoption of mechanical tamping or packing with the shovel, becomes increasingly general.

A third, and more serious, drawback than many ascribed to concrete sleepers, is the feeble transverse stability, of the line which has been noted in certain cases. There is no doubt that the concrete sleeper, because of its hardness and its surfaces, which are generally smooth, has less grip on the ballast than the wooden sleeper. But it will be seen that this fault, instead of being general to the concrete type of sleeper, is only present in particular types of sleepers. The lack of grip between the concrete sleeper and the ballast can be overcome however by appropriate means, which give this sleeper not only equal, but undoubtedly superior, stability to that of the wooden sleeper. It has been objected that the concrete sleeper, on account of its hardness, breaks up the underlying ballast. However, this particularity is very rarely mentioned in the reports published by Railway Companies or private concerns making experiments. Our experience has been that the crumbling effect is barely noticeable. On the other hand, it should be noted that the best foundation for the concrete sleeper is, without doubt, fine ballast mixed with some sand. Certain types are even laid on a layer of sand added to the ballast. The crumbling of the ballast directly under the sleeper, can thereby only ensure a better bearing by making the distribution of the load more uniform without intefering with the standard of packing. When taking up track laid on wooden sleepers, it is found that, under the best packed sleepers which have moved least, the ballast is compressed into a solid bed composed of the usual ballast as it were cemented together by smaller material from either the original ballast or the breaking up of parts thereof.

A final drawback to concrete sleepers, and perhaps the only one of much importance, is the rigidity a track laid with concrete sleepers usually shows. This drawback is particularly noticeable on those lines over which express trains run at high speeds. It is a characteristic feature of the concrete sleeper which cannot possess the elasticity of the wooden sleeper. However, experience has shewn that the limited elasticity of the sleeper cannot in any way influence the safety and regular running of trains. The track assumes the characteristic properties of a *harder* track, but the difference is hardly perceptible. The passage of trains at high speed from a section of track with wooden sleepers on to another laid on concrete, is not made evident by greater oscillations or more violent shocks at the rail joints; the only difference noticeable is a change in the noise produced by the tyres rolling on the rails, which on concrete sleepers has a higher pitch and gives a more metallic note.

On the other hand, the increase in the rigidity of the track affects the method of fixing the rails to the sleepers and affects the sleepers themselves, so that in almost all types of concrete sleepers a more or less large elastic packing is introduced between the rail or sole plate and the bearing surface on the sleeper.

It will be seen further on that in the case of a certain type of sleeper it is possible to increase the elasticity of the track as a whole.

From what has been already said, it will be seen that in general the most important defects of concrete sleepers are really of little importance, and are certainly not such as to give rise to any apprehension as to the use of the type of sleeper which ever it may be that gives every guarantee as regards the essential feature, namely, *length of service*.

Having enumerated in detail the most important disadvantages of concrete sleepers compared with wood the advantages that the former have over the latter must now be considered. These advantages are far from being negligible.

By its nature, the concrete sleeper may be said to be unaffected by atmospheric action. A concrete sleeper, properly designed, theoretically will last for ever, and the cost of upkeep will be very much less than for wooden sleepers.

Further, the fact that concrete sleepers will not burn is a point worth considering, especially where steam traction is used, in particular near stations and in shunting yards, etc.

Another great advantage of the concrete sleeper is that it is possible to manufacture it on sites near where it is to be used; thereby either eliminating the cost of transport or certainly reducing it considerably. There should be no difficulty in finding a site for each zone or division near to a quarry or a stream where a plant can be laid down. This plant can either be permanent for a regular supply for normal demands, or temporary for unusual demands.

Another advantage is the possibility of repairing the concrete sleepers when they have been damaged by an accident or by carelessness on the part of those laying the track, etc.

Passing now to the investigation of the most interesting points of concrete sleepers, the reports read at the International Railway Congress held in Rome in 1922 must be taken into account. As the causes of failures will be looked into, certain types of sleepers which have been discarded will also be mentioned.

All concrete sleepers consist of two main parts; the body of the sleepers (which we will call the sleeper), and the rail fastening. First of all the different types of sleeper will be considered, and then the numerous types of rail fastening.

Types of sleepers. — Whilst wooden sleepers used both by large and small railway systems are, for equal and similar work, of almost the same dimensions, the many varied types of concrete sleepers are of very varied dimensions, the reason being that because of the many difficulties and uncertainties arising in the design of concrete sleepers, those experimenting have preferred to use their own ideas based on intuition and practical experience, which evidently differ to a great extent. These varied types can be classed from the point of view of the sleeper into three main classes :

a) Sleepers capable of resisting bending moments both at the middle and under the rails;

b) Sleepers with the centre section reduced and so incapable of resisting central bending moments;

c) Sleepers consisting of blocks resisting bending moments at the rails, and having a more or less elastic connection, together with a reduction of the bending moment at the centre.

Sleepers of the types *a*) and *b*) are made of reinforced concrete alone, and those in group *c*) are made of reinforced concrete and iron bars.

Sleepers of the first group are evidently designed directly from wooden sleepers, the makers having preserved the traditional form and only modified it very slightly. But it will be seen that the statical properties of concrete sleepers are very different from those of wooden sleepers because of the lack of elasticity of the former. It will be seen that concrete sleepers of uniform section should have very large dimensions to resist the worst conditions of loading.

It must now be noted that the majority of designs of sleepers of group *a*) have too small a section and reinforcement and that the bad results which they have given ought to have been foreseen.

One of the first types of these sleepers is the well known reinforced concrete sleeper with which the Italian State Railways have experimented on a large scale. The sections at the centre and under the rails are shewn in figure 1. The reinforcement is uniform throughout and is made up of 5 bars of 9 mm. (0.354 inch) in diameter for the lower face, and 3 bars of 9 mm. (0.354 inch) in diameter for the upper. The lower bars have a section of about 318 mm^2 (0.493 square inch) and the upper of about 191 mm^2 (0.296 square inch). The sectional area of the concrete is about 200 cm^2 (31 square inches) at the centre and 290 cm^2 (45 square inches) under the rails. The length of the sleeper is 2.60 m. (8 ft. 6 1/2 in.) and the volume is 60 dm^3 (2.12 cubic feet). This type of sleeper was adopted by the Italian State Railways after other types with a much smaller section of the reinforcing bars had been tried, and also after trials had been carried out in the laboratory. The very high proportion of cement used in the mixture for the construction of these concrete sleepers (750 kgr. per cubic metre = 46.8 lb. per cubic foot) should be noted. The results obtained with this sleeper are known to have been to a large extent bad; nearly all the 300 000 sleepers laid down in 1908 had to be withdrawn in a few years.

The result of the figures given above is that in the sleeper in question the greater importance had been given to the lower reinforcement. On the other hand, another type of sleeper may be mentioned, the Percival, of American manufacture in which the greatest attention has been given to the upper reinforcement (fig. 2). The Percival sleeper is of equal section at the centre and under the rails, the upper reinforcement bars having a cross section of about 390 mm^2 (0.604 square inch) and the lower barely 280 (0.434 square inch). This sleeper was used on the « Florida East Coast Railway » and the results were not very satisfactory. The length of the sleeper was 2.40 m. (7 ft. 10 1/2 in.).

Another type of sleeper of type *a*) which may be chosen from among the many types experimented on, is the sleeper supplied to the Swedish Railways, by the « Asbeston-Gesellschaft » of Berlin (fig. 3). This sleeper seems to be well designed with a fairly strong upper reinforcement at the centre and a lower reinforcement under the rails. The section of concrete has an area of about 340 cm^2 (52.7 square inches) at the centre and is increased to 396 cm^2 (61.4 square inches) at the rails. The section of iron in the lower reinforcement is about 200 mm^2 (0.310 square inch) at the centre point and about 400 mm^2 (0.620 square inches) at the rails. The upper reinforcement has, on the other hand, a section at the centre and under the rails of 300 mm^2 (0.465 square inch) and 100 mm^2 (0.155 square inch) respectively.

A characteristic feature is the frame of angle iron under the rails. The length of the sleeper is 2.70 m. (8 ft. 10 1/2 in.).

As we have said, this type of sleeper was tried on the Swedish Railways in 1917. A few months after they had been laid, the sleepers had already started to crack badly in the centre and under the rails. At the end of about one year they had to be withdrawn from use.

The most interesting types of sleeper of this group are the Calot types, in use on the Paris-Orleans Railway.

Ignoring the rail fastening, which will be considered later, the sleeper itself, without having any special features, is remarkable for its simplicity and its very large dimensions as compared with other similar types. It is probable that it is only when made in such sizes that sleepers of this pattern can give sufficiently satisfactory results. The results given by the Calot sleeper can be said to be satisfactory, and this sleeper now forms part of the standard equipment of the Paris-Orleans Company. This railway company uses three patterns of the Calot sleeper : the sleeper for main lines, the semi-reinforced sleeper, and the sleeper for secondary lines.

Figure 4 shews the dimensions for a main line sleeper. The reinforcement (upper and lower) is composed of six rods 9 mm. (0.354 inch) diameter running the full length of the sleeper.

There is also at the centre a supplementary upper reinforcement of 5 bars of the same diameter, and under the rails a lower further reinforcement of 6 round bars of 9 mm. (0.354 inch) diameter.

At the centre, the cross sections of the lower reinforcement bars is 382 mm^2 (0.592 square inch) and of the upper 700 mm^2 (1.085 square inches) and under the rails an upper reinforcement of 382 mm^2 (0.592 square inch) and a lower of and 763 mm^2 (1.183 square inches) section.

The cross section of concrete at the centre and under the rails is 480 cm^2 (74.4 square inches) and the length of the sleeper 2.40 m. (7 ft. 10 1/2 in.).

The comparison of these figures and those of other types of sleepers of the same group is sufficient to explain the more satisfactory results given by this sleeper. For example, the volume of the Calot sleeper is about double that of the previously mentioned sleeper used by the Italian State Railways.

Among other examples of sleepers of this type, considering the sleeper itself alone, the following are worthy of attention, either because of their constructional features or because of the results obtained when tested.

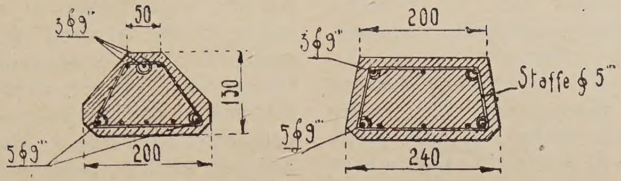

Fig. 1.

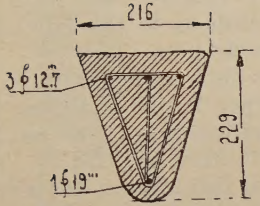

Fig. 2.

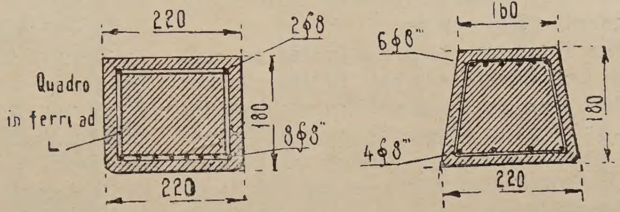

Fig. 3.

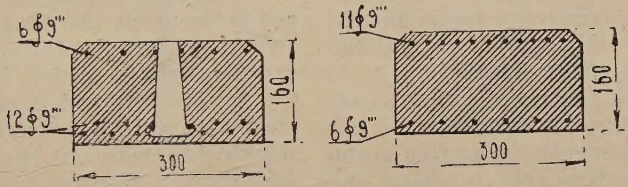

Fig. 4.

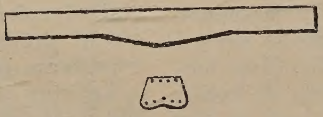

Fig. 5.

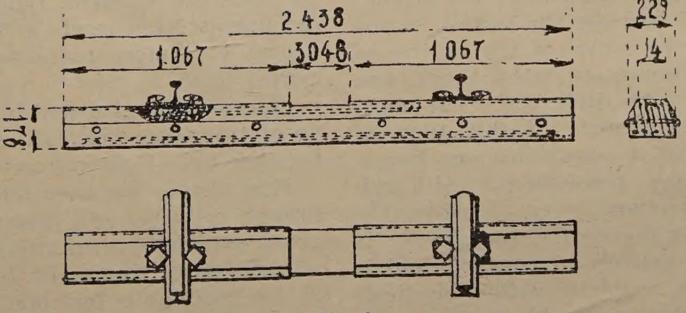

Fig. 6.

Explanation of Italian terms : Quadro in ıerri ad L = Angle iron frame. — Staffe = Bars.

The *Borini* sleeper (fig. 5) is being tried on the Reggio-Guastalla railway. This sleeper has a characteristic profile, similar to that of a solid body having equal strength throughout. To quote the inventor, this shape not only gives a greater power of resistance to the centre section, but also does away with excessive packing of the centre portion, with the consequent increase of bending moment at the centre.

The *Rognoni* sleeper, similar in section and reinforcement to the Italian State Railways' sleepers, already described, but having a special form of fastening (see description further on), tested on a fairly large scale on the North of Milan Railway.

The *Righetto-Zorzi* sleeper : in spun reinforced concrete being tried on the Milan-Gallarate tramway; distinguished by the method of manufacture and the concave profile of the sleeper.

In the case of sleepers coming under the second group *b*), makers have not tried to give the sleepers a section at the middle sufficiently large to stand the normal bending moments, but instead have tried to reduce, as far as possible the value of these bending moments. For it is well known, as will be shown later on, that the greatest bending moments in the centre are set up by excessive packing at the centre part of the sleeper as compared with that at the ends. They have therefore tried logically enough to increase the section of the ends and reduce that of the centre, so that the phenomenon stated above, can only occur with difficulty. The most characteristic feature of this type of sleeper is that it constitutes one homogeneous piece throughout. Although the cross sections vary considerably starting from the centre, the sleeper is nevertheless entirely of concrete and is consequently, unable to deflect elastically in any way comparable with the deflection of wood sleepers.

First of all, although not specifically coming under group *b*), mention may be made of the Atwood sleeper, which in reality is a sort of transition between the two groups. Great attention is given in the design of this sleeper to the reinforcement of the ends, as shewn in figure 6. Sleepers of this pattern were laid about 1918 on the Pittsburgh and Lake Erie Railway, and were still in service in 1921, with satisfactory results. We have no further information as to the latest behaviour of these sleepers.

Another more interesting example of this type of sleeper is that tested by the Italian State Railways in 1918 on the line from Rome to Florence (fig. 7). The central portion of this sleeper has a very reduced cross section, calculated to a very limited pressure on the ballast. As shewn in figure 7, the ends of the sleeper are made up of two blocks which alone should support and transmit the pressure exerted by the rolling load. However, the results obtained during ten months' trial were so little satisfactory that it was decided to take out the 2 000 sleepers laid down. Further to the serious defects in the rail fastening, the sleeper itself developed serious defects. At the end of a few months, the track became out of gauge owing to breakages in the transverse rods.

Leaving out of account the rail fastening, and only examining the most characteristic examples, mention must be made of the Vagneux type, constructed throughout in concrete and being a step nearer to a more perfected design discussed later.

Figure 8 shews that this type is made up of two large prismatic blocks joined together by a light concrete beam.

This sleeper has been tried by various French railways and tramways and has given satisfaction, particularly on the smaller gauge tramways, but was found to be insuitable for main lines over which heavy and fast trains run.

Sleepers of the third group, derived from those of the second, retain the lar-

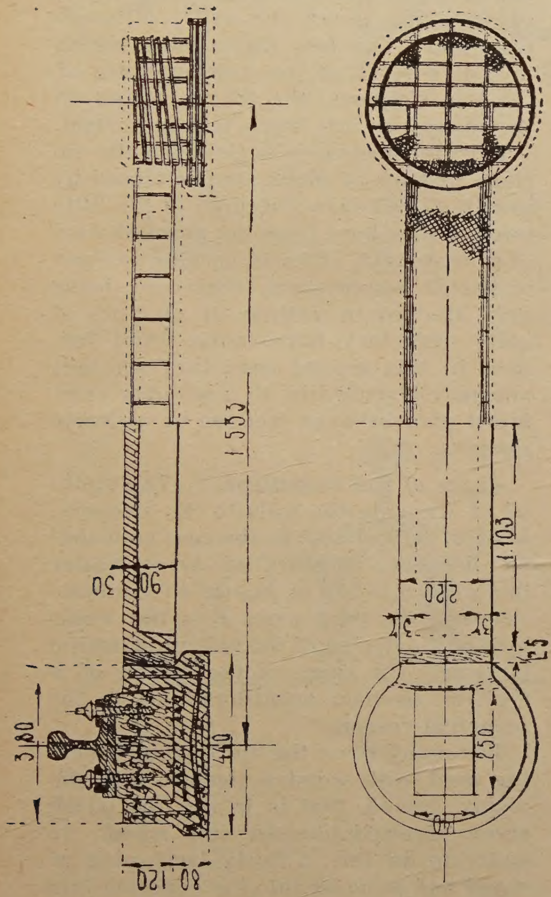
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Fig. 7.

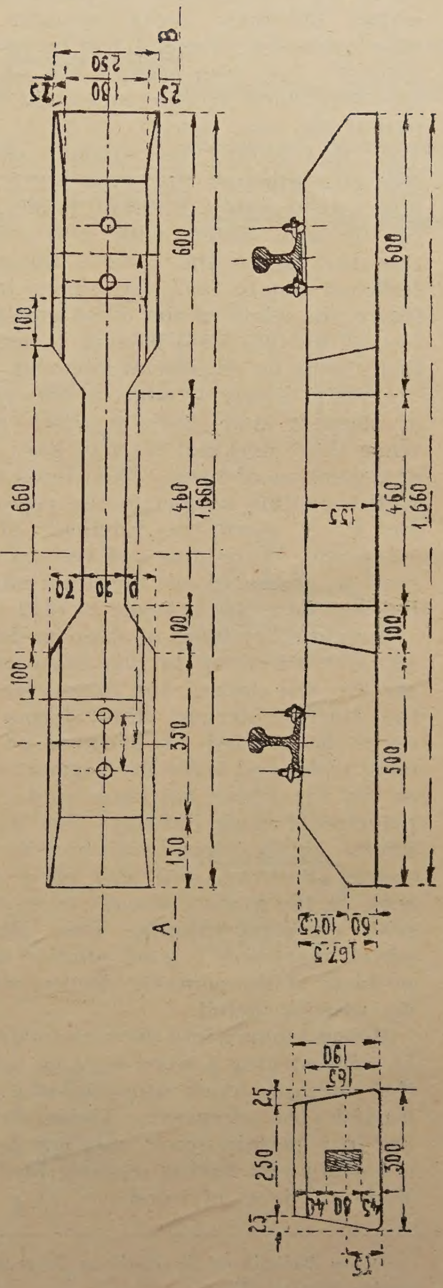
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Fig. 8.

gely developed ends acting as supports whilst the beam of concrete joining them which obviously lacks strength has been discarded, a metal bar being substituted. This iron crosstie bar will be shewn to fulfil another important function at the same time.

In this group, the Kimball sleeper (fig. 9), although the main governing idea was probably different, is mentioned. It was tested with only moderate success on the « Pere Marquette Railroad » up to 1902. As shewn in the figure, the whole of the metal portion is formed by two steel bars of U section, there being no other reinforcement.

Certain results of trials of this group of sleepers date back to 1907, since when there does not seem to have been any mention of them. We would refer whilst on this subject to the report of Mr. C. H. Cartlidge, Engineer of the « Chicago, Burlington and Quincy Railroad » presented at the International Railway Congress at Rome in 1922 (1).

Among other types of sleepers designed, and belonging to this third group, are the Bell sleeper, the Adams sleeper, the Murray sleeper (this sleeper has never been tried in practice on any large scale) and lastly the Vagneux sleeper (fig. 10), which has been tried on the principal French Railways and, in particular, on the Paris, Lyons and Mediterranean Railway, and also on a small scale on the Swiss Railways, everywhere with excellent results. This sleeper, which represents a good solution of the problem of the concrete sleeper, will be discussed in detail.

Before looking into the various designs of rail fastening a word must be said on the old stone block supports (the early English *pot sleepers*). These sleepers are in a certain sense a return to this old equipment having separate blocks of stone or pieces of wood.

If we give up the idea of joining sleepers rigidly together and are content to maintain the gauge by means of tie rods or wooden sleepers laid in at intervals according to a well thought out scheme, it is certainly not difficult to get reinforced concrete supports giving every guarantee as to safety. In fact the problem is now almost reduced to the method of fastening down the rails; although this again has less importance seeing that as a result of the independance of the two supports, the forces acting on the fastenings are much less important.

It is not necessary to describe the various systems of block supports used by nearly all railways : because of the little interest they have from the point of view of construction, and also because of their restricted employment, their use being only allowed in stations or on lines of quite secondary importance. And further, in this second case, they are laid alternately according to a suitably combined plan between wooden or concrete sleepers.

Types of rail fastenings. — The method of securing the rails to the sleepers, of great importance in the case of wooden sleepers, becomes of even greater importance when it comes to concrete sleepers. In most cases it is the weakness of the design of the fastening, which is largely the cause of the failure of a type of sleeper; occasionally it is the principal reason.

For many years the old type of fastening used with wooden sleepers was slavishly copied, that is to say the coach screw screwed likewise into wood. In order to do this, a fairly large plug of wood had to be let into the concrete into which the coach screws could be screwed. Naturally only wood of good quality was used and the connection between the concrete and the wood was made as tight as possible.

A system of this kind was used on the sleepers already described as used on the Italian State Railways. A recess, in

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(1) See *Bulletin of the Railway Congress*, December 1920, p. 815.

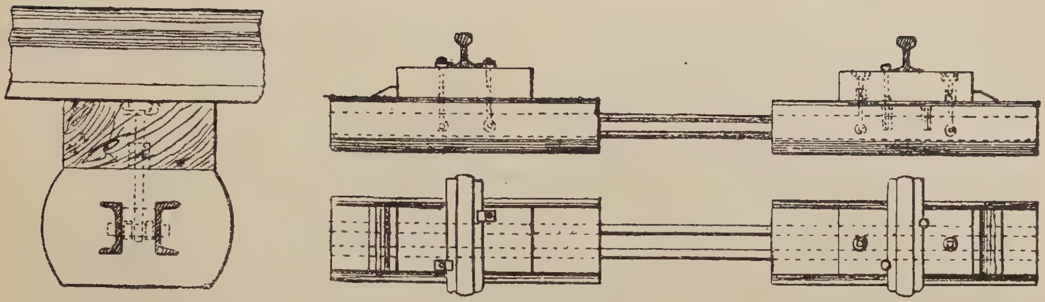

Fig. 9.

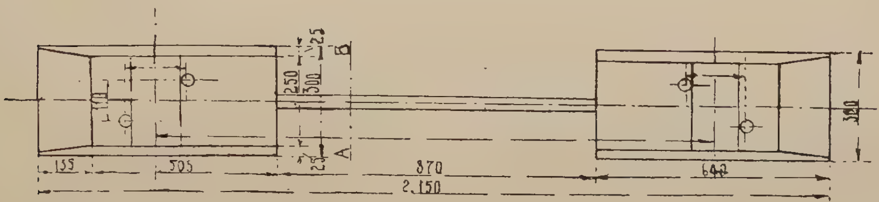

Fig. 10.

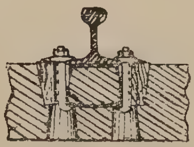

Fig. 12.

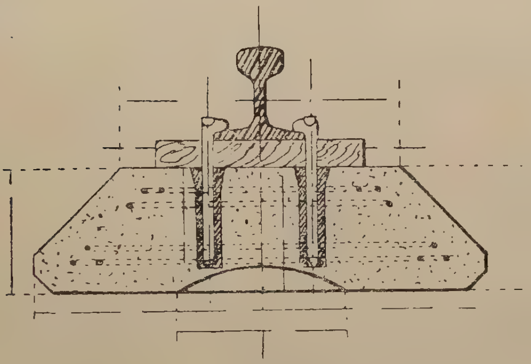

Fig. 11.

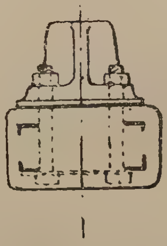

Fig. 13.

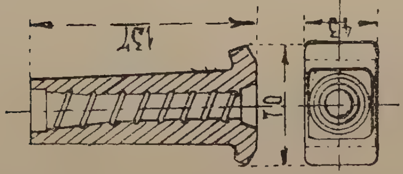

Fig. 14.

Figs. 9 to 14.

the form of a truncated pyramid was cast in the concrete, to take a hard wood plug into which the coach screw would be screwed.

A great number of other types of sleepers were equipped in a similar way. The first patterns of the Borini sleeper, certain types of English block sleepers, having the fastening shewn in figure 11, and the Bates sleeper (fig. 12) tried on the Elgin, Joliet and Eastern Railway, may be mentioned.

All other designers, whilst retaining the principle of the coach screw screwed into wood, adopted much larger dimensions for this wooden plug. Thus in the Kimball sleeper, already described, a large block of white oak protrudes above the top surface of the concrete. Still another type of sleeper using this method is the sleeper used on the Italian State Railways shewn in figure 7; but in this case the wooden plug is completely embedded in the concrete.

However, all these ways of fixing the rails gave in practice bad results, a tight and lasting fit between the wood and the concrete not being obtainable.

Moreover, wood however hard the choice, is too sensitive to atmospheric conditions. The average sized plug always swells during prolonged wet periods and then contracts during the dry. In the first instance, the plug exerts considerable pressure on the surrounding conglomerate and may cause cracks; in the second instance, play between the wood and the concrete arises, and in the end causes a slacking of the fastening as a consequence seriously aggravating the hammering action of the rail on the sleepers. The effects of this hammering action are harmful in the case of the concrete sleeper both because of the direct action of the rail on the top surface of the sleeper, and also because of those actions transmitted to the body of the sleeper through the parts of the rail fastening. In spite of elastic packings placed between the rail and the sleeper, the hammering of the rail or the bearing plate on the surface of the concrete often causes surface disintegration. Sometimes, owing to defective casting or unfavourable conditions of laying, *viz.*, ballast too big, or hard ground, nearness of joints, this disintegration spreads over considerable portions of the sleeper and even lays bare the steel reinforcement.

Even more serious is the action set up by this hammering in the body of the sleeper through slack rail fastenings, as it causes cracks which affect the whole section and can put the sleeper out of use quite a short time after it has been laid.

A second system of rail fastening consists of bolts cast in or fixed by some means into the concrete beam, the rails being fixed by means of nuts. This system, which in certain types of sleepers, appears to be exactly like the attachment as used on metal sleepers, has not been tried to any extent on reinforced concrete sleepers. The system of casting the bolt in the concrete is not to be recommended, as if it deteriorates for any reason it would not be possible to replace it. This is a serious drawback as the bolts are very apt to be bent during handling or laying.

On the other hand, if the bolt is not cast in and openings are provided for the bolts to pass through, very great difficulties arise, especially as regards the distribution of the stresses throughout the body of the sleeper. There are few types of this form of attachment in existence and none of them, so far as we are aware, has given really satisfactory results. However, this may be, the system tested on the North Eastern Railway (fig. 13) and that of the new patterns of the Borini sleeper may be quoted.

On the other hand much better results have been obtained by retaining the holding down bolt as the rail fastening and inserting into the mass of the concrete a screwed ferrule, either completely or partly metal. A notable advantage of this system is that it allows the same holding down screws, already in use for wooden sleepers to be used, without consequently, the various railway companies having to provide and stock new patterns.

It has just been stated that the ferrule can be either entirely or partly metal; in either case it is not possible to cut the thread directly in the concrete. In the Calot sleeper it is wholly metal. This sleeper described above is in use on the Paris-Orleans Railway. The bush is in cast iron (fig. 14) a thread being cast on the inside at the foundry, into which the holding screw is screwed. The outside is cast to the shape that will best guarantee it bonding properly with the mass of the concrete, and at the same time act on the lower reinforcement of the sleeper. The bush is of course placed in the mould before the concrete is poured : once set a really good grip is assured.

The semi-metal bush is nowadays adopted on a large scale by concrete sleeper manufacturers, and is the system which gives the best practical results. This system is really only an application of the well-known Thiollier bush to concrete sleepers.

A Thiollier spiral of suitable dimensions, is screwed on to a coach screw previously coated with oil or soap, set in the mould. The mould having been poured and the concrete set either partially or totally, the coach screw is withdrawn, and the spiral is left in the concrete. This system, probably first used in the Rognoni sleeper, which has already been described, constitutes, with several variations, one of the most interesting points of the Gin sleeper (French Patent No. 455294, fig. 15). This system has been adopted by the Italian State Railways for the concrete blocks, and it is also employed in nearly all the types of the Vagneux sleeper. Figure 16 represents the fastening of a composite sleeper S. C. A. F. T. (Vagneux).

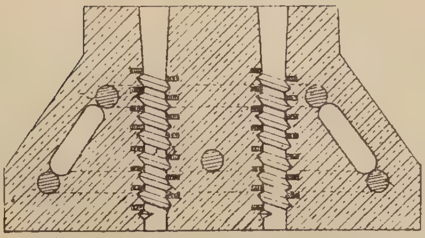

Fig. 15.

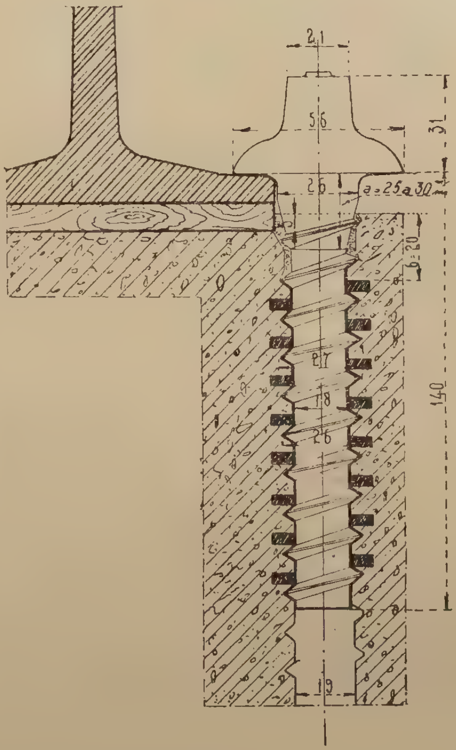

Fig. 16.

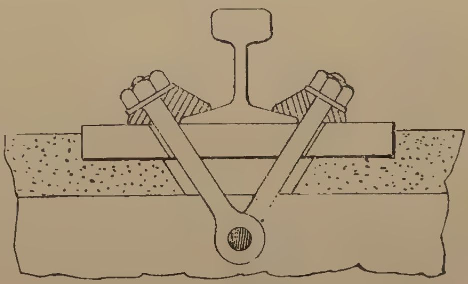

Fig. 17.

The excellent practical results obtained by the use of this system of attachment are well justified, owing to the fact that the forces transmitted by the screw are uniformly distributed over the mass of concrete through the medium of the spiral which has a strong hold on the concrete.

Besides the systems already mentioned, there are many others of entirely different design. Those worthy of mention are the W. J. Bell fastening (American Patent No. 711-277) of quite good design, but according to our opinion it would not give very practical results (fig. 17), and the fastening of the Righetto-Zorzi sleeper which, without acting on the concrete casting as a whole (which would be difficult in view of the tubular shape of the sleeper), simply consists of a metal bar which, by means of bolts, ensures the proper connection of the rail and sleeper.

Before concluding the investigation into the principal parts of concrete sleepers, a word must be said on the shock absorbing material which is placed as a rule between the rail or the saddle and the sleeper. It has already been stated that the reason for this is to render the road or track more elastic. Other advantages are also gained; it absorbs the stresses transmitted by the rail and ensures that there is a uniform distribution of this pressure over the surface of the sleeper. Numerous experiments have been made on this resilient packing. Wood which is the simplest is used in the form of thick or thin pieces. But wood, whether it has been treated or not, has been found unsuitable for this purpose, as at the end of a short period it shrinks and slips from beneath the rail; this is a very serious defect since considerable play is set up between the rail and the holding down screw, with the usual harmful results.

Rubber has been tried as well, but it was not long before it was discarded owing to its feeble resistance to atmospheric conditions, to the easiness with which it hardened, and to its high price.

Linoleum has given quite good results against mechanical action and lasts well. But at the end of about a year it has been noticed that the slabs commenced to disintegrate owing to atmospheric actions. Quite good results have been obtained with wool felt pads, with or without previous soaking in paraffin; they have been found however not to keep in place. The best results obtained so far have been obtained with woods of special kinds, highly compressed before being put into service in such a way as to make the mass of the wood more compact to and render it less sensitive to variations in the atmospheric conditions.

Good results have also been given by the Borini sleeper, in which a wooden packing is enclosed in and protected by a metal fitting which replaces the ordinary sole bearing plate. The Borini sole plate designed for wooden sleepers to save the cost of sole plates, has also been used on concrete sleepers. The Italian State Railways have adopted it for their concrete sleepers, already described, and it is also used on the sleepers designed by Borini; this type has also been described (fig. 5).

Conclusions. — Having described in some detail the most interesting experiments carried out with concrete sleepers, and having examined the main details of the more interesting types, it can be said that certain evident conclusions can be drawn which, as will be seen subsequently are confirmed and proved by theoretical considerations. Taking into account only tests, the results of which are well known and which deal with the use of such sleepers on lines carrying heavy high speed traffic, the following points can be recorded :

The reinforced concrete sleeper formed as a one piece girder, having a simple body with a more or less constant section and with reinforcement bars throughout its length, has given as a rule bad results. Apart from any crumbling away or cracking, due to the imperfect system of rail fastening, more or less serious cracks or fractures have developed at the centre and underneath the rails. It has been noticed that the cracks at the centre were more frequent and more serious in the short sleepers, and always had the appearance of cracks due to insufficient resistance of the sleepers to the bending moments at the centre.

The cracks under the rails were more frequent and serious on the bottom surface (negative bending moment) of the longer sleepers, in this case being due to insufficient resisting power, although they were frequently confused with cracks caused by the system of rail fastening. Leaving the fastening out of the question, the only way to obtain a sleeper of this type which will stand up consists in suitably increasing the cross section and the reinforcement to the great detriment, it must be admitted, of the economical aspect of the concrete sleeper. As has been seen already the Calot sleeper has been designed on these lines.

Concrete sleepers, composed of two blocks joined together by a beam of small section also of concrete, have given bad results (breakages at centre); that is to say, the reduction of the bending moments at the centre, due to the reaction at the centre being diminished, is not sufficient to justify the reduction of the section opposing these moments, seeing that the sleeper continues to function as a rigid girder.

The composite sleeper of steel and concrete has probably given the best practical results and will, when well designed, solve the problem definitely.

The system of attaching the rails to the sleeper which has given the best results up to date is that formed by the Thiollier fastening cast in the concrete.

For every type of sleeper, the great importance, from the point of durability, of careful construction and good quality of material should be noted. More than once practical tests have shewn that groups of sleepers, identical and laid in lines having the same traffic conditions give very different results.

It should be noted that tests carried out with a limited number of sleepers are not very convincing either because the inspection and maintenance are in such cases too well carried out or because on a short length of line it is not possible to know the unforetellable conditions, of particular importance, under which the stresses in the sleepers will act.

Lastly, the considerable importance of the kind of ballast cannot be overlooked as it has considerable effect on the lasting properties of the concrete sleepers. Fine ballast, which ensures an even distribution of the loads, gives the best practical results.

PART II.

1. Part I of this article briefly considered the principal types of reinforced concrete sleepers experimented with up to the present, dividing them into three categories :

a) Sleepers in the form of a girder of uniform section;

b) Sleepers with a reduced cross section in the middle;

c) Composite concrete and steel sleepers, or pot sleepers.

As has already been said, the second category of sleepers has only been used in rare instances and has given little satisfaction. It does not, therefore, seem necessary to consider them in detail, and we will confine ourselves to a study of the characteristics of the uniform girder sleeper and the pot sleeper.

We all know that the permanent way does not lend itself readily to an exact mathematical investigation. The actual working conditions of the track depend on extremely variable elements which cannot easily be expressed in simple formulæ, amongst which may be mentioned the difficulty of evaluating the dynamic stresses transmitted by the rolling stock, the variations in the intensity of shock at the rail joints, the irregularity of the reaction of the ballast on the sleeper, depending either on the nature of the formation and of the ballast itself or on the tamping of the sleepers or, in general, on the state of repair of the permanent way.

In the following study the dynamic stresses, among other factors, will not be taken into account so that the conclusions come to will have no claim to complete accuracy. It should be stated, however, that the hypotheses advanced (they are the classic hypotheses of the German treatises) are sufficiently accurate, whether for calculations regarding concrete sleepers, or, and more particularly, for comparing the relative behaviours in use of the timber sleeper, the concrete girder sleeper and the composite sleeper.

Experiments and practical experience have not always confirmed the hypotheses referred to above; thus, as regard deflection they have almost always given inferior results to those obtained by calculation. This fact is not, however, sufficient to deprive the theory of Winkler and Zimmermann of its value, since the discrepancy between the experimental results and the theoretical figures is generally attributed to the imperfect determination of the coefficient of ballast, a coefficient which seems in practice to attain values appreciably higher than the maximum of 8 postulated by Zimmermann for permanent ways with and without ballast. As we shall point out later, we have in the present study increased the maximum value of the coefficient of ballast from the traditional figures of 3 and 8 to 4 and 10 respectively.

When considering reinforced concrete sleepers on the basis of the Winkler hypothesis we shall add a word on certain particular cases of stressing, especially in order to give an idea of the nature and degree of the accidental stresses which may affect the sleeper, and in particular the capacity of the various types of sleepers to resist these stresses.

Finally, we shall revert to, and make a more detailed comparison between, the girder sleeper and the pot sleeper, by contrasting their different characteristics.

2. *Hypotheses and methods of calculation.* — As already mentioned, in our examination of sleepers we have applied the formulæ of Zimmermann, Hoffmann, Schwedler, etc., and adopted as basis the fundamental hypothesis of Winkler.

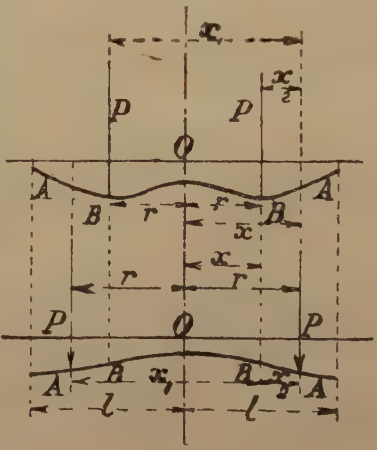

Fig. 18.

The Winkler hypothesis, which is well confirmed by practical experience, assumes that the ballast behaves as an elastic body and, within the limits of ordinary stresses, is uniformly elastic. If we represent the live load by p and the deflection of the ballast by y, we have :

$$p = Cy \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where C is a coefficient which is assumed to depend solely on the nature of the ballast and of the formation, and to which has been given the name « coefficient of ballast ». If p is expressed in kilogrammes per square centimetre and y in centimetres, C is expressed in kilogrammes per cubic centimetre, and represents the unitary pressure to which the ballast must be subjected to give an elastic flexure of one centimetre.

It will be remembered that, starting from this hypothesis, Zimmermann has developed an ingenious theory regarding the railway sleeper, arriving at the formulæ which we reproduce below (¹).

Giving the letters the signification which they have in figure 18, and, further, writing :

C = the coefficient of ballast,
E = the modulus of elasticity,
I = the moment of inertia of the sleeper,
b = the width of the sleeper,
y = the deflections of the ballast,
M = the moments of the sleeper,
P = the load imparted by the rail,
p = the unitary pressures on the ballast,

and taking :

$$X = \sqrt[4]{\frac{Cb}{4EI}}\,; \qquad \lambda = Xl\,; \qquad \rho = Xr\,; \qquad \xi = Xx \quad . \; . \; . \; . \quad (2)$$

we have in general :

$$y = \frac{XP}{Cb}\left|\frac{1}{2}(\eta_1 + \eta_2) + u_\rho \cdot ch\,\xi \cos\xi + v_\rho\, sh\,\xi \sin\xi\right| \quad . \; . \; . \; . \; . \quad (3)$$

$$M = \frac{P}{2X}\left[\frac{1}{2}(\mu_1 + \mu_2) - v_\rho \cdot ch\,\xi \cos\xi + u_\rho\, sh\,\xi \sin\xi\right] \quad . \; . \; . \; . \; . \quad (4)$$

where

$$\left\{\begin{aligned} \eta_1 &= e^{-\xi_1}(\cos\xi_1 + \sin\xi_1)\,; \\ \eta_2 &= e^{-\xi_2}(\cos\xi_2 + \sin\xi_2)\,; \end{aligned}\right. \qquad \left\{\begin{aligned} \mu_1 &= e^{-\xi_1}(\cos\xi_1 - \sin\xi_1) \\ \mu_2 &= e^{-\xi_2}(\cos\xi_2 - \sin\xi_2) \end{aligned}\right. \quad . \; . \; . \; . \quad (5)$$

and

$$\left\{\begin{aligned} u_\rho &= u\, ch\,\rho \cos\rho + u\, sh\,\rho \sin\rho \\ v_\rho &= v\, ch\,\rho \cos\rho + v\, sh\,\rho \sin\rho \end{aligned}\right. \qquad (6)$$

and

$$u = \frac{2 + \cos 2\lambda - \sin 2\lambda + e^{-2\lambda}}{sh\,2\lambda + \sin 2\lambda}$$

$$v = \frac{\cos 2\lambda + \sin 2\lambda - e^{-2\lambda}}{sh\,2\lambda + \sin 2\lambda} \qquad (7)$$

$$w = \frac{2 - \cos 2\lambda + \sin 2\lambda - e^{-2\lambda}}{sh\,2\lambda + \sin 2\lambda}$$

Distinguishing, by means of the indices o, ρ, λ, or o, r, l, between the relative values at the middle of the sleeper, at the rail seat and at the extremity of the sleeper respectively, and representing by $[\eta]$ and by $[\mu]$ the terms between brackets in formulae (3) and (4), we have :

$$\begin{aligned} &[\eta_o] = \eta_\rho + u_\rho\,; \qquad [\mu_o] = u_\rho - v_\rho \\ &[\eta_\rho] = [\mu_o] + ch\,\rho \cos\rho - [\mu_o]\, sh\,\rho \sin\rho \qquad (8) \\ &[\mu\rho] = [\eta_\rho] + ch\,\rho \cos\rho + [\eta_\rho]\, sh\,\rho \sin\rho \end{aligned}$$

and, to summarise :

Middle section :

Deflection of the ballast : $y_o = \dfrac{XP}{Cb}[\eta_o]$

Unitary pressure : $p_o = \dfrac{XP}{b}[\eta_o]$ (9)

Bending moment : $M_o = \dfrac{P}{2X}[\mu_o]$

(¹) H. Zimmermann : *Die Berechnung des Eisenbahn-Oberbaues* (Permanent way calculations). Berlin, 1888.

Rail seat :

Deflection of the ballast : $y_r = \frac{XP}{Cb} [\eta_\rho]$

Unitary pressure : $p_r = \frac{XP}{b} [\eta_\rho]$ (10)

Bending moment : $M_r = \frac{P}{2X} [\mu_\rho]$

End section :

Deflection of the ballast : $y_e = \frac{XP}{Cb} [\eta_\lambda]$

(11)

Unitary pressure : $p_e = \frac{XP}{b} [\eta_\lambda]$

Bending moment : Nil.

In all the above equations the value of P, that is to say, the maximum load which the rail transmits to the sleeper is assumed to be known. If we indicate by G the wheel load, that is to say the load transmitted to the rail, the value of the ratio $\frac{P}{G}$ must naturally depend either on the nature and strength of the rail, or on the distance from one sleeper to another, or on the elastic properties of the ballast and sleeper.

A rail is generally considered as a continuous girder on elastic supports; it is thus that it is regarded by numerous authors who have more or less limited the length of the girder and put forward various hypotheses on the distribution of the loads.

Let E represent the unit of elasticity of the steel of the rail, I the moment of inertia of its cross section, and a the distance between centres of two consecutive sleepers. Let us assume further that :

$$B = \frac{6EI}{a^3}; \quad D = \frac{Cb}{X[\eta_\rho]}; \quad \gamma = \frac{B}{D} \quad (12)$$

Hoffmann's formula, derived from the hypothesis expressed by figure 19, can be stated thus :

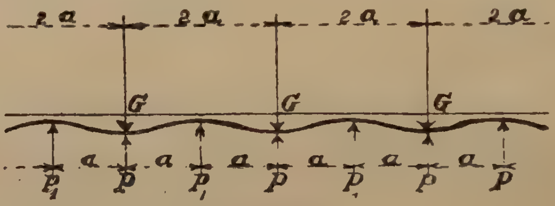

Fig. 19

$$P = \frac{4\gamma + 1}{8\gamma + 1} G \quad . \quad . \quad . \quad (13)$$

Schwedler's formula, derived from the hypothesis expressed by figure 20, becomes :

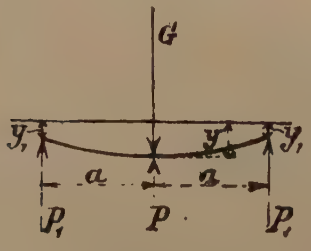

Fig. 20.

$$P = \frac{\gamma + 2}{3\gamma + 2} G \quad . \quad . \quad . \quad (14)$$

Comparatively recently, Mr. R. Desprets has taken up the study of the determination of rail tensions (¹), regarding the rail as a continuous girder of indefinite length and always resting on elastic supports. The values of the ratio $\frac{P}{G}$ obtained by him are lower than those

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(¹) R. Desprets : « Rail calculations. » (*Bulletin of the Railway Congress*, August 1921.)

resulting from the formulæ of Hoffmann, Schwedler and Zimmermann, and, consequently approach more closely to the results obtained in practice. The practical application of the Desprets theory is, however, very laborious, therefore, in view of the nature of our present investigation, we consider it sufficient to avail ourselves of the above formulæ, as a rule using the equation suggested by Schwedler (14).

It may be noted, further, that the study of sleepers can also be represented graphically by methods of « false position » (1) the analytical method described, whilst apparently very laborious, is nevertheless greatly facilitated by the numerous numerical tables calculated on the Zimmermann formula (2), and consequently we shall adopt this method in comparing the timber sleeper with the concrete girder sleeper and the composite sleeper.

3. *The equi-strength girder.* — We shall indicate by the words « equi-strength girder » the reinforced concrete girder which, being within the allowable dimensions for use as a railway sleeper, can stand, with an equal guarantee of safety, the same bending moments as the timber sleeper.

Naturally, the mere fact of equal resistance is not sufficient to determine the cross section of the sleeper, as one can conceive of an infinity of girders of this nature. It may, however, be stated that :

a) The height of the girder is necessarily very limited, firstly so as not to increase the quantity of ballast necessary for laying the track, and secondly so as not to diminish the already limited elastic properties of the girder;

b) The width also is restricted to definite limits : where very wide sleepers are used, the free space between adjacent sleepers is naturally very reduced, and consequently they are difficult to tamp. The smaller the distance between the sleepers (or in other words the greater the importance of the line) the more serious becomes this disadvantage.

The limitation of the height is particularly serious, because the concrete girder sleeper will always be uneconomic from the point of view of making the best use of the material in it; and, consequently, as we shall see later, the steel will always be lightly stressed.

In the case of the timber sleeper, assuming it to be of oak or other hard timber, with a breaking strength under bending of 600 kgr. per cm² (8 520 lb. per square inch), it may be taken, at a prudent estimate, that the safety load is equal to 100 kgr. per cm² (1 420 lb. per square inch).

The moment of resistance of an ordinary sleeper of the dimensions given in figure 21 is

$$W = \frac{1}{6} bh^2 = \frac{1}{6} \cdot 24 \times 14^2 = 784 \text{ cm}^3$$
(47.8 cubic inches).

Consequently, the moment M which the sleeper is capable of supporting with the necessary degree of safety will be :

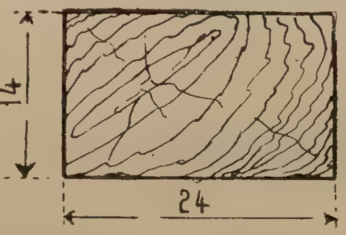

Fig. 21.

$$M_s = WK = 784 \times 100 = 78\,400 \text{ kgr.-cm.}$$
(68 500 inch-pounds).

In the case of the reinforced concrete sleeper, assuming that good concrete of a mixture of not less than 300 kgr. (19 lb. per cubic foot) can give under compression a breaking strength of 225 kgr. per cm² (3 200 lb. per square inch), and assuming the prescribed coefficient of safety to be equal to 4, the maximum unitary stress will be 56 kgr. per cm² (796 lb. per square inch).

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(1) G. Guidi : *Esercizi sulla scienza delle costruzioni.* Turin, 1923.

(2) Zimmermann : *op cit.*

It should be noted that this stress value in concrete, already somewhat high in itself, is perhaps excessive for the girder sleeper. The sleeper is in effect a girder with a very limited cross section, and consequently any imperfection or lack of uniformity in casting makes a serious difference. If this fact necessitates great accuracy in construction, it also, however, imposes a prudent limitation of the value of the unitary stresses. In our opinion the stress value in the conglomerate of the sleepers, as shown by calculation, should not exceed 40 kgr. per cm² (568 lb. per square inch).

If, however, we consider that the cross section of the timber sleeper, shown in figure 21, is a theoretical cross section, whereas in practice appreciable reductions are admissible which increase still further with wear and the destruction of the sapwood, and that in consequence, in wood also, the stresses corresponding to a bending moment of 78 400 kgr.-cm. (68 500 inch-pounds) will be in practice appreciably higher than the value of 100 kgr. per cm² (1 420 lb. per square inch) indicated above, we shall adhere to the stresses of 100 and 56 adopted for wood and concrete respectively, considering that they afford in practice approximately the same degree of safety in the two types of sleeper.

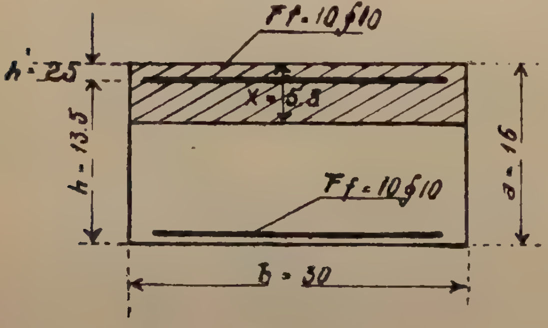


Fig. 22.

As we have already said, there is no need to go into the question of stress values in the steel, as these are very low.

The cross section which we have contemplated for the equi-strength girder sleeper is that shown in figure 22. The height of 16 cm. (6 5/16 inches) appeared the greatest allowable for laying in the permanent way and for ensuring a not too excessive increase in rigidity, while the width of 30 cm. (11 13/16 inches) cannot be appreciably increased if, with a limited space between adjacent sleepers, the tamping is not to be rendered too difficult.

It should be noted that this cross section of 30 × 16 cm. (11 13/16 × 6 5/16 inches) corresponds to a cross section already adopted for reinforced concrete sleepers, and in particular for the Calot sleeper, which is used with satisfactory results on the Paris-Orleans Railway. Once the cross section has been decided upon, the reinforcement of the sleeper is thereby automatically determined.

The position of the neutral axis is given, according to Mörsch, by the formula :

$$X = -\frac{m\,(F_f + F'_f)}{b} + \sqrt{\left[\frac{m\,(F_f + F'_f)}{b}\right]^2 + \frac{2m}{b}\,hF_f + h'F_f)}$$

where F_f and F'_f are the cross sections of the lower and upper reinforcements, m is the ratio between the modulus of elasticity of steel and concrete (= 10), and the other letters have the same signification as in figure 22.

In our case, where the reinforcement is symmetrical, the value of X becomes :

$$X = -\frac{2mF_f}{b} + \sqrt{\left(\frac{2mF_f}{b}\right)^2 + \frac{2mF_f}{b}(h+h')}$$

whence :

$$X = -\frac{2\times 10\times 7.85}{30} + \sqrt{\left(\frac{2\times 10\times 7.85}{30}\right)^2 + \frac{2\times 10\times 7.85}{30}(13.5+2.5)} = 5.3$$

and the stresses in the concrete :

$$\sigma_c = \frac{M}{\frac{bx}{2}\left(h-\frac{x}{3}\right) + mF_f\frac{x-h'}{x}(h-h')}$$

$$= \frac{M}{\frac{30\times 5.3}{2}\left(13.5-\frac{5.3}{3}\right) + 10\times 7.85\times\frac{5.3-2.5}{5.3}(13.5-5.3)} = \frac{M}{1\,390}.$$

For the maximum moment $M_s = 78\,400$, we have :

$$\sigma_c = \frac{78\,400}{1\,390} = 56.3 \text{ kgr. per cm}^2 \text{ (745 lb. per square inch),}$$

and the stresses in the steel :

$$\sigma_f = m\sigma_c\frac{h-x}{x} = 10\times 56.3\times\frac{13.5-5.3}{5.3} = \text{about 870 kgr. per cm}^2 \text{ (12 370 lb. per square inch).}$$

$$\sigma'_f = m\sigma_c\frac{x-h'}{x} = 10\times 56.3\times\frac{5.3-2.5}{5.3} = \text{about 300 kgr. per cm}^2 \text{ (4 260 lb. per square inch).}$$

As will be seen, this type of sleeper satisfies fairly closely the desired equivalence of strength.

It should be remarked that we have adopted equal upper and lower reinforcements, whereas it would have been logical and economic to allow for a dissymmetrical reinforcement, that is to say, to increase the upper steel and to reduce the lower in the middle section, and correspondingly to reduce the upper steel and increase the lower in the section at the rail seat.

In view, however, of the short length of the sleeper, the economy realisable by varying the distribution of the reinforcement would be very small; further, it should be remembered that the sleeper may also be subjected, particularly in the middle section, to dangerous inversions of moments, due to particular conditions of distribution of the reactions.

In any case, an unsymmetrical distribution of the reinforcement can only modify to what may be considered a negligible extent the results which will be obtained relative to the elasticity of the equi-strength girder.

4. *Comparison between the elasticity of the timber sleeper and that of the equi-strength concrete sleeper.* — We will now compare the elasticity, and thereby the stress values, of the timber sleeper and the equi-strength concrete sleeper respectively. In order that the comparison may be as complete as possible, we must not confine ourselves to a given length, but consider all the lengths which are possible in practice.

We have already seen that the ratio between the maximum load transmitted by the rail to the sleeper and that transmitted by the wheel to the rail depends not only upon the nature of the sleeper, but also upon the dimensions of the rails and, above all, on the pitch of the sleepers. For the purpose of clarity and simplicity we will choose a particular type of permanent way, *viz.* the type RA 36, which has a sleeper pitch of 83 cm. (2 ft. 8 5/8 in.).

It must be noted, however, that the results obtained are of a general character, the influence exercised by the variations in the type of permanent way on the ratio of the moments and deflections as between the timber sleeper and the concrete sleeper being very slight.

We will now see what values are given by the Schwedler formula for the type of permanent way just mentioned. We have already given the value of

$$\frac{P}{G} = \frac{\gamma + 2}{3\gamma + 2},$$

where

$$\gamma = \frac{B}{D} = \frac{6EI}{a^3} : \frac{Cb}{X[\eta_\rho]}.$$

For the permanent way RA 36, with $a = 83$, we have :

$$B = \frac{6EI}{a^3} = \frac{6 \times 2\,000\,000 \times 1\,016}{83^3} = 21\,331.$$

The values of

$$X = \sqrt[4]{\frac{Cb}{4EI}}$$

are independent of the length of the sleeper and of the nature of the permanent way. They are exclusively functions of the nature and form of the section of the sleeper, and of the quality of the ballast and of the formation. We have already defined the equi-strength types of timber and concrete sleepers; the values of C being fixed, we can calculate that of X.

As we have already remarked, Zimmermann has taken the limits of value of the coefficient of ballast as 3 and 8 : the numerous experiments since carried out have shown that the effective deflections were appreciably lower than those arrived at by calculation, and the reason for this difference has been attributed largely to the unduly low values given to the above mentioned coefficients. In the present article we shall, therefore, adopt the values C = 4 and C = 10, as we consider that they correspond sufficiently closely to ordinary ballast conditions. As regards the modulus of elasticity, we shall adopt the values of 110 000 and 150 000 kgr. per cm², respectively for timber and concrete (¹).

For the timber sleeper we shall have, therefore :

$$I = \frac{1}{12} bh^3 = \frac{1}{12} \times 24 \times 1^3 = 5\,488 \text{ cm}^4, \text{ and}$$

$$\text{for } C = 4 \ldots X = \sqrt[4]{\frac{24 \times 4}{4 \times 110\,000 \times 5\,488}} = 0.0141.$$

$$\text{for } C = 10 \ldots X = \sqrt[4]{\frac{24 \times 10}{4 \times 110\,000 \times 5\,488}} = 0.0178.$$

(¹) The figure of 150 000 adopted for the modulus of elasticity of reinforced concrete is certainly less than the real value. Our only reason for adopting it is that it is laid down by the official regulations relating to reinforced concretes.

For the concrete sleeper we shall have :

$$I = I_b + 10 I_f.$$

$$I_b = \frac{1}{12} bh^3 = \frac{1}{12} \times 30 \times 16^3 = 10\,240 \text{ cm}^4.$$

$$I_f = 7.85 \times \frac{11^2}{2} = 475.$$

$$I = 10\,240 + 4\,750 = 14\,990 \text{ cm}^4,$$

and then :

$$\text{for } C = 4 \ldots X = \sqrt[4]{\frac{30 \times 4}{4 \times 150\,000 \times 14\,990}} = 0.0108.$$

$$\text{for } C = 10 \ldots X = \sqrt[4]{\frac{30 \times 10}{4 \times 150\,000 \times 14\,990}} = 0.0135.$$

For the ordinary gauge, with which alone we are concerned, we shall have, further :

$$C = 4 \ldots \begin{cases} \text{timber sleeper } \rho = X_r = 0.0141 \times 75 = 1.058, \\ \text{concrete sleeper } \rho = 0.0108 \times 75 = 0.81; \end{cases}$$

$$C = 10 \ldots \begin{cases} \text{timber sleeper } \rho = 0.0178 \times 75 = 1.33, \\ \text{concrete sleeper } \rho = 0.0135 \times 75 = 1.02. \end{cases}$$

As regards the length of sleepers, in order to make the comparison more complete we have taken into consideration lengths from 2 to 3 metres (6 ft. 6 3/4 in. to 9 ft. 10 1/8 in.), which certainly represent the limits of length for sleepers on lines of normal gauge. The values of $\lambda = X\,l$ thus established, it has been possible to calculate the value of

$$[\eta_o],\ [\eta_\rho],\ [\mu_o],\ [\mu_\rho],\ [\eta_\lambda],$$

and therefrom the values of $\gamma = \frac{BX[\eta_\rho]}{Cb}$,

then the values of

$$\frac{P}{G} = \frac{\gamma + 2}{3\gamma + 2}$$

and finally the values of the moments, of the deflections and of the pressures in the middle, rail seat, and end sections of the sleeper, with a given load G. In these calculations the value of G has been taken as equal to one ton; consequently, in order to ascertain the corresponding values for any given load, all that is necessary is to multiply the above values by the number of tons per wheel. From these values diagrams have been prepared which give a sufficiently clear picture of the relative behaviour of the two types of sleeper.

The curves in figure 23 (based on a coefficient of ballast of C = 4) represent the bending moments at the middle and at the rail seats, as well as the unit deflections and pressures at the rail seats. The moments are given in kilogrammetres per tonne of wheel load, the deflections in 100ths of a millimetre, and the pressure in kilogrammes per cm², in each case for a wheel load of one tonne.

As will be seen, the bending moments which occur in the equi-strength concrete sleeper are practically always higher than those which occur in the timber sleeper.

As regards the bending moments at the rail seats, the difference is very slight up to a length of about 2.50 m. (8 ft. 2 1/2 in.) after which it increases as the length increases. With a length

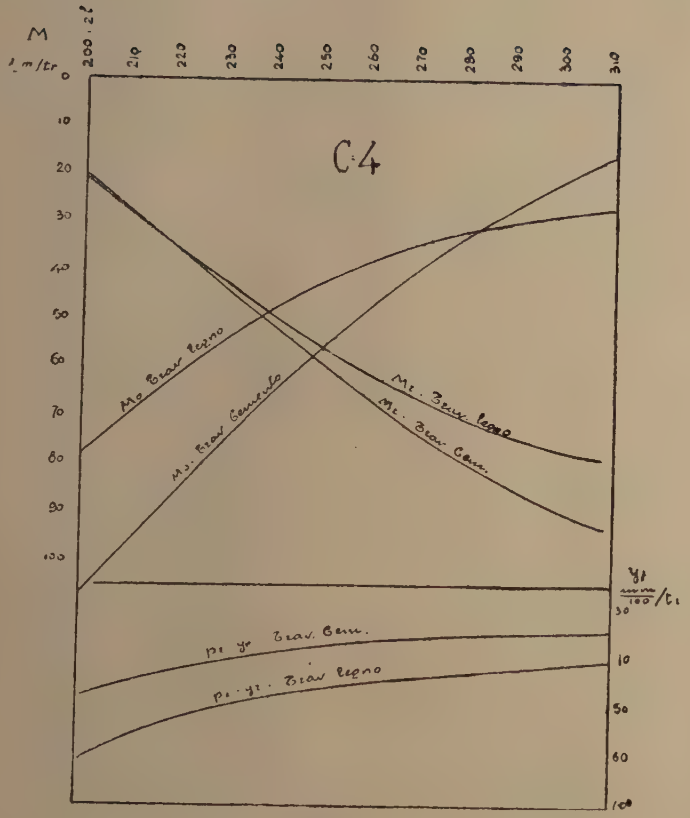


Fig. 23.

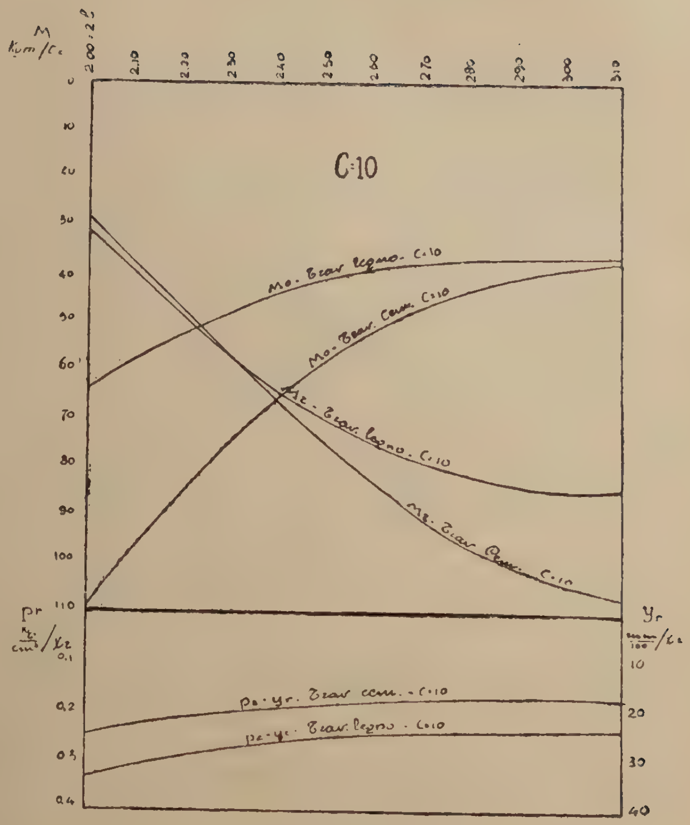


Fig. 24.

Explanation of Italian terms : Trav. cemento = Concrete sleeper, — Trav. legno = Timber sleeper, of 3 m. (9 ft. 10 1/8 in.), the bending moments at the rail seats are about 20 % higher in the equi-strength sleeper than in the timber sleeper.

On the other hand, the difference is much greater as regards the moments of the middle section : whereas for a sleeper 2.80 m. (9 ft. 2 1/4 in.) long the moments are identical for both timber sleeper and concrete sleeper, for a length of 2.50 m. (8 ft. 2 1/2 inch.) the moments of the concrete sleeper are about 30 % higher, and for lengths of 2.20 m. (7 ft. 2 5/8 in.) and under, about 40 % higher.

Naturally, in the timber sleeper the deflections and, consequently, the pressures, are greatest at the rail seats (maximum values).

The curves of figure 24 represent graphically the same values, but based on coefficient of ballast of C = 10. The difference between the values of the moments is naturally greater.

There is an identical bending moment at the rail seats for a sleeper length of 2.30 m. (7 ft. 6 1/2 in.), for a length of 2.60 m. (8 ft. 6 1/2 in.) the increase is about 12 %, rising to about 25 % for a length of 3 m. (9 ft. 10 1/8 in.)

For the bending moment at the middle section, the increase is 7 % for a length of 3 m., 30 % for a length of 2.60 m. and about 60 % for a length of 2.20 m.

In order to give a clearer idea of the extent of the difference in behaviour, we have assumed, for the sake of example, a sleeper subjected to loads of 20 t. per axle, or 10 t. per wheel. We have then calculated — always in relation to lengths — the maximum unitary stresses in timber and in concrete, and we have represented them graphically in the curves of figures 25.

It appears from this diagram that a timber sleeper of the dimensions stipulated would afford the necessary guarantees of safety for all lengths between 2 m. (6 ft. 6 3/4 in.) and 2.70 m. (8 ft. 10 1/2 in.), that is to say, for all practicable lengths, while the lengths admissible for the equi-strength concrete sleeper would lie between 2.30 m. and 2.50 m. (7 ft. 6 1/2 in. and 8 ft. 2 1/2 in.)

5. *Conclusions relative to the behaviour of the girder sleeper under the Winkler hypothesis.* — From what has been said we may draw several conclusions regarding the behaviour of the girder sleeper, examined on the basis of the Winkler hypothesis.

In general, a girder sleeper, although capable of resisting with equal guarantees of safety the same bending moments as a timber sleeper, does not in fact offer the same guarantees of safety, since in view of its lesser elasticity it is subjected to greater bending moments. In order to obtain the required guarantee of safety it would be necessary, therefore, to increase the dimensions and the reinforcement of the girder. As, however, an increase in the width of the sleeper is not admissible beyond certain limits (30 to 35 cm. [12 to 14 inches]), it would be necessary to increase the height and the reinforcement and, in consequence, to increase the rigidity of the girder. This would cause a still further increase in the bending moments of flexure and consequently the reinforcement of the cross section would have to be still greater.

Further, it should be noted that the section contemplated is already much stronger than those adopted up to the present for concrete sleepers (except in the case of the Calot sleeper), and that an increase of the reinforcement would be increasingly uneconomic from the point of view of the metal in the girder, while an increase in height would not only have a great effect on the rigidity, but would entail greater expense in laying the track in view of the greater quantity of gravel that would be necessary. A further increase in the dimensions of the girder would, therefore, lead to unduly high unitary weights. The equi-strength sleeper, having a length of

2.50 m. (8 ft. 2 1/2 in.), weighs as it is about 290 kgr. (640 lb.), that is to say four times the weight of an equivalent timber sleeper.

Nor is it desirable to reduce the cost and weight of the sleeper by reducing the length, for as we have seen the moments at the middle increase in this case much more rapidly than in the case of the timber sleeper. This is, moreover, completely borne out by experience, which has shown that breakages and deterioration of reinforced concrete sleepers with a uniformly shaped girder always occur at the rail seats in the case of long sleepers and in the middle in the case of short sleepers.

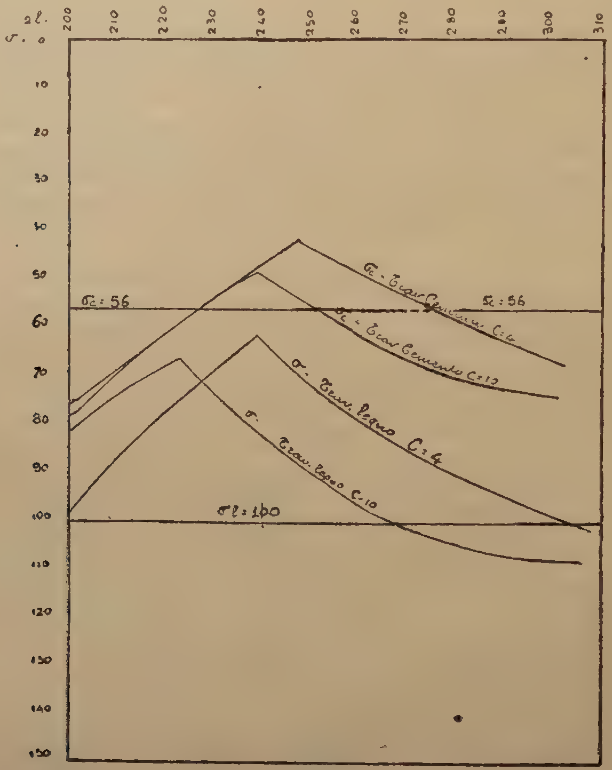


Fig. 25.

Explanation of Italian terms : Trav. cemento = Concrete sleeper.
Trav. legno = Wood sleeper.

In view of these considerations, and of the high cost which would be entailed by the use of concrete sleepers constructed in accordance with such criteria, it seems permissible to conclude that up to the present the concrete sleeper with a uniformly shaped girder scarcely forms an economic substitute for the timber sleeper.

6. *Behaviour of the composite sleeper under the Winkler hypothesis.* — As we have already remarked, the mixed sleeper consists of two blocks of reinforced

concrete, connected by a steel rod of a relatively small cross section. The presence of this connection, whilst fulfilling the function of giving the sleeper the features of a complete whole, cannot, under load, appreciably modify the stresses which develop in the blocks, and, therefore, for the purposes of calculation, it is reasonable to consider the blocks themselves as isolated, and, as in the hypotheses referred to in the case of the girder sleeper, to assume that the ballast is uniformly elastic. We shall see that the deflections of the block are so small that it can be considered as absolutely rigid, or, in other words, the value of the unitary reaction of the ballast may be considered as constant. In this case, which is borne out by experience, the axis of the mixed sleeper does not undergo any deformation under the action of an excess load, but is displaced parallel to itself.

We shall see, however, that the action of the connection comes into play when certain particular conditions of stress occur.

For the moment, therefore, the block of the composite sleeper will be considered as an isolated block, that is to say, as a support subjected to a load which we shall assume to be always applied along the axis, as there is no reason to contemplate blocks subjected to unsymmetrical loads.

The formulæ of Zimmermann relative to the isolated support (always on the Winkler hypothesis, P = y C) are as follows :

$$y_o = \frac{P}{D} = \frac{XP}{2Cb}[\eta_o]; \qquad p_o = \frac{XP}{2b}[\eta_o]; \qquad M = \frac{P}{4X}[\mu_o] \quad . \; . \; . \; . \; \text{(15)}$$

where the dimensions are those of figure 26, and where the other letters have the same signification as we have given them up to the present.

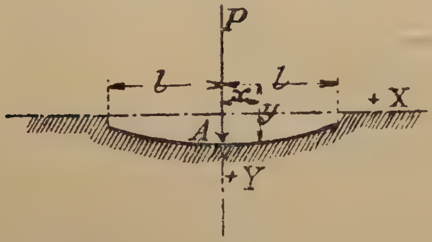


Fig. 26.

In these expressions we have :

$$\left.\begin{aligned}[\eta_o] &= 1 + u = \frac{2 + ch\,2\lambda + \cos 2\lambda}{sh\,2\lambda + \sin 2\lambda} \\ [\mu_o] &= 1 - v = \frac{ch\,2\lambda - \cos 2\lambda}{sh\,2\lambda + \sin 2\lambda}\end{aligned}\right\} \text{(16)}$$

The value of the ratio $\frac{P}{G}$ will be calculated by means of the formulæ :

$$\frac{P}{G} = \frac{\gamma + 2}{3\gamma + 2} \quad . \; . \; . \; . \; \text{(14')}$$

where

$$\gamma = \frac{B}{D} \text{ et } D = \frac{2Cb}{X\,(\eta_o)}.$$

If, instead of applying the Zimmermann formulæ, we considered the block as rigid, we should have :

$$y_c = \text{constant} = \frac{P}{2Cbl},$$

$$p_c = \text{constant} = \frac{P}{2bl}, \qquad M_c = \frac{Pl}{4}$$

and the ratios between the preceding values and the present values would be :

$$\frac{y_o}{y_c} = \lambda\,[\eta_o], \qquad \frac{p_o}{p_c} = \lambda\,[\eta_o], \qquad \frac{M_o}{M_c} = \frac{\mu_o}{\lambda} \quad \text{(17)}$$

These ratios presuppose that P, and therefore $\frac{P}{G}$, is constant, which in fact is not the case. Considering the block as rigid, we have :

$$Dc = 2Cbl$$

and then :

$$\frac{D}{D_c} = \frac{1}{\lambda\,[\eta_o]} \text{ and, therefore, } \frac{\gamma}{\gamma_o} = \lambda\,[\eta_o].$$

As will be seen, in our case the values of

$$\lambda\,[\eta_0] \text{ and of } \frac{[\mu_0]}{\lambda}$$

are very near to unity.

Passing to the application of the formulæ stated above, let us assume the cross section of the block to be that already contemplated for the equi-strength concrete girder sleeper, and let us again take the two coefficients of ballast as C = 4 and C = 10. We will examine the lengths of block which can be used in practice, that is to say for a total length of sleeper of from 2 to 2.50 m. (6 ft. 6 3/4 in. to 8 ft. 2 1/2 in.). A shorter length would not be admissible, while a greater length, by reducing unduly the length of the connecting bar, would deprive the composite sleeper of its principal characteristics. These two limits of sleeper length correspond to block lengths of 50 cm. and 1 m. (1 ft. 7 11/16 in. and 3 ft. 3 3/8 in.), *i. e.* l = 25 and l = 50. For the values of l comprised between these two limits, the maximum bending moments of flexure at the middle have been calculated in each case for a load of one ton per wheel.

The values of these moments, when C = 4 and C = 10, have been represented graphically in the curves of figure 27, where we also show the curves representing the maximum moments of the timber sleeper and the equi-strength concrete sleeper.

As will be seen from the diagrams, for C = 4 the curve of the maximum bending moments in the block sleeper intersects the curve for the timber sleeper at a point corresponding to a sleeper length of about 2.35 m. (7 ft. 8 1/2 in.), and the curve for the concrete sleeper at a point corresponding to a length of 2.45 m. (8 ft. 1/2 in.). Where C = 10, the lengths at which there is equality of moments are respectively 2.20 m. (7 ft. 2 5/8 in.) and 2.37 m. (7 ft. 9 9/16 in.). This means that under the most unfavourable conditions the maximum bending moments of the block sleeper are lower than those of the girder sleeper for lengths less than 2.37 m. (7 ft. 9 9/16 in.), and that they are lower than those of the timber sleeper of equi-strength cross-section for lengths less than 2.20 m.

This length of 2.20 m., which corresponds to a block length of 70 cm. (2 ft. 3 1/2 in.), is precisely that adopted in the S. C. A. F. T. sleeper. It is clear, moreover, that it is not advisable to reduce this length, as this would entail the risk of increasing the pressure on the ballast and the deflection, which appear already sufficiently high.

Before concluding our examination of the behaviour of the block sleeper under the Winkler hypothesis, we will point out the very slight influence exercised by the deflection of the block on the value of the stresses sustained. If we consider the block as perfectly rigid and, consequently, the reaction under the supporting surface as uniformly distributed, for

$$2l = 55 \qquad C = 4 \qquad \lambda = 0.3$$

we shall have :

$$D = 2\,Cbl = 2 \times 4 \times 30 \times 27.5 = 6\ 600$$

$$\gamma = \frac{B}{D} = \frac{21\ 331}{6\ 600} = \simeq 3.23$$

$$\frac{P}{G} = \frac{\gamma + 2}{3\gamma + 2} = \frac{5.23}{11\ 69} = \simeq 0.448$$

and for

$$2l = 92 \qquad C = 4 \qquad \lambda = 0.3$$

$$D = 2 \times 4 \times 30 \times 46 = 11\ 040$$

$$\gamma = \frac{21\ 331}{11\ 040} = 1.93$$

$$\frac{P}{G} = \frac{3.93}{7\ 79} = 0.503$$

On the other hand, applying the Zimmermann formulæ, we have :

for $2l = 55 \qquad C = 4 \qquad \lambda = 0.3$

$$D = \frac{2Cb}{X(\eta_0)} = \frac{240}{0\ 0108 \times 3.3387} = \simeq 6\ 660$$

$$\gamma = \frac{21\ 331}{6\ 660} = \simeq 3.2$$

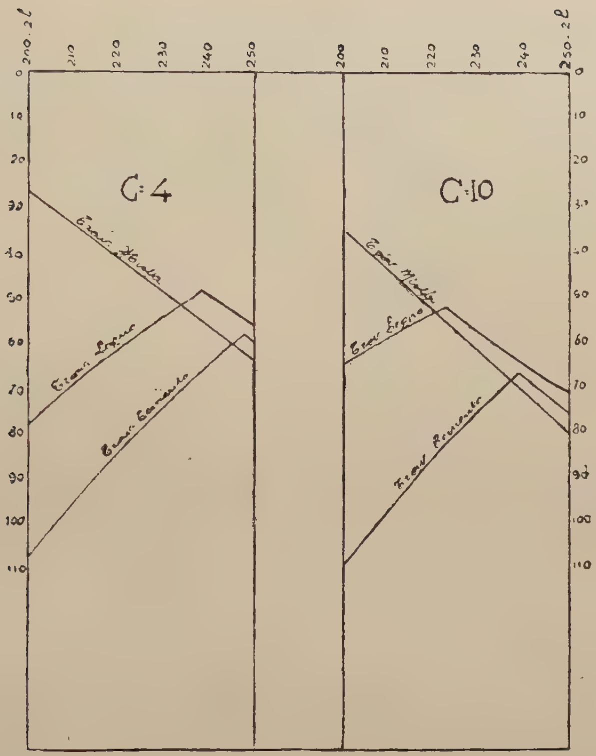


Fig. 27.

Explanation of Italian terms : Trav. cemento = Concrete sleeper. — Trav. legno = Wood sleeper. Trav. mista = Composite sleeper.

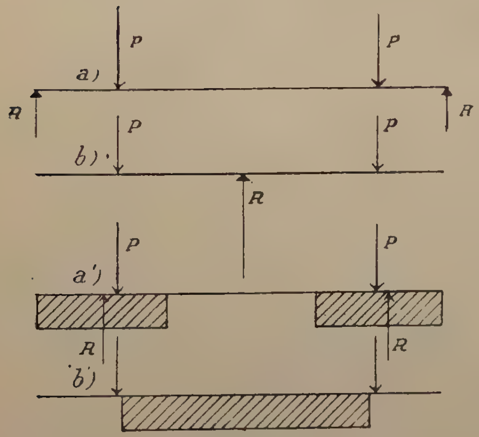
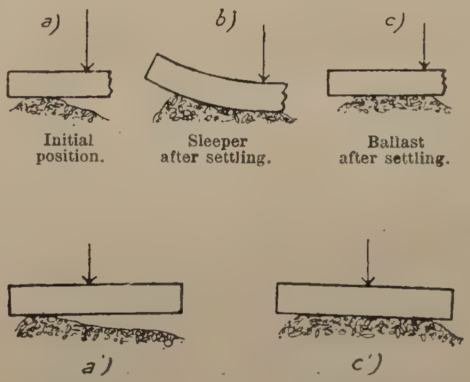


Fig. 28.



Initial position.

Sleeper after settling.

Ballast after settling.

Initial position.

After settlement.

Fig. 29.

$$\frac{P}{G} = \frac{5.2}{11.6} = \simeq 0.448$$

and for $2l = 92$ $C = 4$ $\lambda = 0.5$

$$D = 10\,970$$
$$\gamma = 1.95$$
$$\frac{P}{G} = 0.501$$

It will be seen that the values $\frac{P}{G}$ are practically equal. The ratio between the value of deflections, the pressures and the moments in the two cases will, therefore, be given by the formulæ (17), which give the following values :

for $\lambda = 0.3$ . . . $\frac{y_o}{y_c} = \frac{p_o}{p_c} = \lambda\,(\eta_o) = 1.001$ $\frac{M_o}{M_c} = \frac{\mu_o}{\lambda} = 0.999$

for $\lambda = 0.5$ . . . . $\frac{y_o}{y_c} = \frac{p_o}{p_c} = 1.012$ $\frac{M_o}{M} = 0.994$

This quasi-identity between the relative values confirms what we have previously asserted, *viz.* that if the isolated blocks are considered under the Winkler hypothesis, the action exerted by the connection, on which naturally no reaction can be exerted, is entirely negligible.

7. *Particular conditions of stress in the girder sleeper.* — As we have already remarked on several occasions, the Winkler hypothesis assumes that the characteristics of the ballast and the formation (ground, packing, etc.) are constant under the whole bearing surface of sleeper. This assumption, which is necessary for the purpose of arriving at a reasonably definite calculation, is, of course, never entirely realised in practice. The actual conditions will approach more or less closely to the assumed theoretical conditions in proportion to the degree of perfection of maintenance of the permanent way, and the degree of uniformity of the ballast and of the soil below. Consequently, although the Winkler hypothesis is in general sufficiently approximate for the examination of a sleeper, and still more for comparing two different types of sleeper, account must be taken of the particular stresses to which the sleepers themselves may be subjected. In our case we shall not attempt to arrive at numerical results nor make too detailed a study, for this would be not only extremely difficult but also probably useless, in view of the impossibility of establishing exactly the values of the most unfavourable conditions of stress experienced in practice. We shall confine ourselves, therefore, to examining in a general manner these stresses, and in particular the capacity of the various types of sleeper to support them and to delimit them.

The most unfavourable conditions of load to which a girder sleeper can theoretically be subjected are those indicated in figure 28 by the letters *a*) and *b*), *viz.* reaction concentrated at the extremity of the sleeper and reaction concentrated at the middle. These cases are purely theoretical, and no sleeper in use up to the present, whether of wood, iron or reinforced concrete, would be capable of resisting the corresponding moments. With an axle load of 16 tons, *i. e.* with $C = 8$, even only assuming that $P = 0.5$, $G = 4$ tons, and assuming a sleeper 2.60 m. in length, we should have in the first case a maximum bending moment $M = 220\,000$ kgr.-cm. (190 960 inch-pounds), and in the second case $M = 300\,000$ kgr.-cm. (260 400 inch-pounds). These moments would give rise in the ordinary sleepers to stresses approaching or exceeding the breaking stress. It should be noted that in case *a*) the value of the moment depends on the length of the sleeper, whereas in case *b*) the same moment is quite independent of the length and depends solely on the pitch, which we have always assumed to be normal.

Corresponding to the theoretical conditions a) and b), there are in practice the conditions a') and b'), also represented in figure 28. If it is assumed in fact that by reason of a particular inequality in the tamping, or for some accidental cause, the theoretical conditions may exist when laying a sleeper or before the passing of the first train, the application of the live load will have the following consequences :

1. The ballast, subjected to pressure, will yield either elastically or in a permanent manner, because of the disintegration of its elements and of their sinking into the permanent way formation, in such a manner as to provoke reaction over a greater surface of the formation.

2. The sleeper also, on undergoing an elastic deformation, will tend to affect a larger surface of the ballast.

The combined effect of these two factors is to transform the conditions a) and b) into those denoted by a') and b'). Assuming equality of initial conditions, the reaction will tend to spread the more uniformly in proportion as the elasticity of the ballast and that of the sleeper are greater.

We have here a practical realisation of the theoretical hypotheses; but conditions a') and b') may also occur normally, at any rate within certain limits, in an ordinary permanent way.

The tamping is generally only done under the ends of the sleepers, on the inside and outside of the track, to a distance varying from 40 to 60 cm. (16 to 24 inches) from the axis of the rail. This tamping under the ends naturally tends to reduce considerably the participation in the reaction of the area of ballast underlying the middle of the sleeper. When (with very thorough tamping) this reaction may be considered as nil, the girder sleeper functions in the same manner as a block sleeper. When, however, for some reason or other the tamping is not symmetrical in relation to the axis of the rail, we revert to stress condition a') and the resultant of the reaction on one end of the sleeper will find itself more or less displaced from the axis of the rail.

The continual passing of trains and their hammering action result in a certain settling of the ballast, with a loosening of the ballast at the sleeper ends and a somewhat rapid increase of the reactions at the middle of the sleeper. During this period, which is the longest, the sleeper finally assumes practically the conditions contemplated in the Winkler hypothesis.

After a certain period (normally one or two years), the loosening of the ballast at the sleeper ends and the over-tamping of the middle section are greatly accentuated. The sleeper finally works almost exclusively on its middle section, thus realising condition b'). This condition manifests itself at the surface of the covering ballast in a manner familiar to the permanent way staff, that is to say, by the loosening of the ballast itself, due to the oscillation imparted to the sleeper by the passing trains. This state of affairs which, while dangerous from the point of view of safety of the service, is generally without serious consequences for timber sleepers, is, on the other hand, almost always fatal to reinforced concrete girder sleepers, which invariably split in the middle.

From what has been said above it follows that, even under particular conditions of load, due either to accidental factors or to the natural deterioration of the permanent way, the elasticity of the sleepers is of considerable importance from the point of view of their strength, for the capacity of adaptation to the irregular surface of the ballast is one of the principal factors in a uniform distribution of the reaction, and consequently limits the accidental value of the bending moments. It should be noted that these moments may assume, for each section, either positive or negative values.

Further, the concrete equi-strength girder sleeper, being much more rigid than the timber sleeper, will be so much the more subjected to suffer from irregular stress conditions, which result in this sleeper in stresses appreciably higher than those which occur in the timber sleeper.

Although it is not possible to estimate exactly, or even approximately, the difference between the relative values, an attempt has been made to show their varying behaviour by inserting in figure 30 the elasticity curves of the two types of sleeper under conditions of stress of type *a'*), that is to say, provoking an inversion of the moments in the middle section of the sleeper. In order to render the case more general, it has been assumed that, substituting the resultant of the reactions for the reactions themselves, and considering exclusively the part of the sleeper comprised between the rail axes, the eccentricities of the resultants of the reactions have been of such a nature as to give rise to the greatest possible bending moment compatible with sleeper safety. This moment which, as we have seen, is equal to about 80 000 kgr.-cm. (69 400 inch-pounds), will be constant for the whole length under consideration. The elasticity curve of the timber sleeper is denoted in the figure by the letter *c*, and that of the equi-strength girder by the letter *a*.

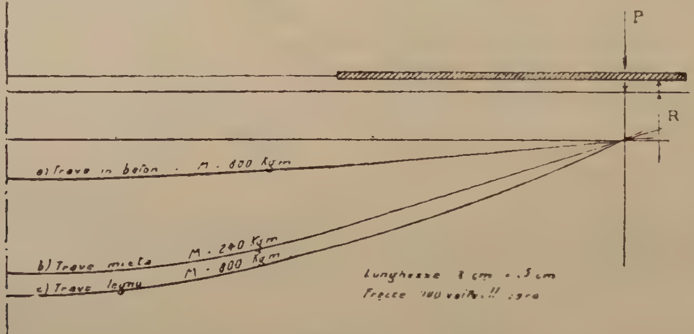

P
R
a) Trave in beton . M . 800 Kgm
b) Trave mista M . 240 Kgm
c) Trave legno M . 800 Kgm
Lunghezze
Frecce

Fig. 30.

Translations of legend : Length : 1 cm. = 75/8 cm. — Deflections : 45 times actual value

Nothwithstanding what has been said above, the elasticity of the sleeper must not be excessive, as this might result in irregular settlement of the ballast, altering the cant of the rails and the gauge. We shall deal with this point in more detail later.

8. *Particular stress conditions in the composite sleeper.* — We have seen (§ 6) that, in the Winkler hypothesis, the connection between the two blocks is subjected to negligible stresses. The composite sleeper might, in these conditions, simply be replaced by two blocks connected together by a rod, the role of which would consist solely in resisting horizontal forces and maintaining the gauge of the line.

These disadvantages presented by isolated blocks are, however, well known, and we know also how difficult it is to maintain the isolated blocks in a horizontal position. With the isolated block, the process of ballast adjustment, referred to in § 7, is in fact lacking. In order to make this fact clearer, let us compare the behaviour of the complete sleeper and of the isolated block in the case which occurs when, by reason of the difference in tamping or of the different nature of the ballast material, or of the difference in resistance of the underlying soil, it is a part of the ballast outside the middle of the rail which reacts either alone or to a large extent. (Initial position *a*), figure 29.)

When the live load is applied, the complete sleeper is deformed elastically and the ballast is also deformed, either elastically or permanently, until the position of equilibrium is attained, assuming, of course, that this position can be attained before the limits of strength of the sleeper are exceeded. During this period the greatest pressures which are exerted on the different zones of ballast will be the greater according as their superelevation is more pronounced: there will occur in consequence a kind of settlement, consisting of the sinking or disintegration of the displaced elements. It is evident, then, that this settlement will be the greater as the measure in which the deflection of the sleeper contributes towards the increase of the surface of reaction is smaller, or, in other words, as the sleeper is more rigid. The repetition of these actions gives rise in a very short time to the settlement of the ballast shown in figure 29*c*).

The isolated block, on the other hand, if placed under the same initial conditions, will tend, on the application of the live load, to assume immediately the position shown in figure 29*c'*), the deflection arising from the twisting of the rail being quite negligible, particularly in view of the imperfect behaviour of the fastenings. Whenever, therefore, the imperfect level of the surface of equilibrium depends on the lack of uniformity of the underlying soil, or on the varying nature of the elements of the ballast, the tilting of the block will tend continually to increase.

If, on the one hand, in order that the sleeper as a whole may not be subjected to too great bending moments, it is essential that it should be sufficiently elastic, on the other hand it is essential, for the proper preservation of the level of the track, of the cant of the rails and, consequently, of the gauge, that a certain rigidity be maintained.

What are the elastic properties which the sleeper must possess from this double point of view ? It would not be possible to determine them theoretically, but long experience has shown that the ordinary timber sleeper adequately fulfils the two contradictory conditions which we have just mentioned.

It would seem desirable, therefore, that the supporting blocks should be connected, not by a simple rod, but by a girder capable of giving to the whole unit elastic properties which shall not be too different from those of the timber sleeper.

The connecting girder, constituting with the two blocks one complete whole, guarantees a resistance to all secondary and accidental actions which is much greater than that of the isolated blocks.

As we have said, the dimensions of the connecting girder must, more than anything else, be determined by practical experience. With the composite sleeper, with which we shall deal below (S. C. A. F. T.), with a total length of 2.20 m. (7 ft. 2 5/8 in.) and, consequently, a free length of connection of 80 cm. (2 ft. 7 1/2 in.), the best results have been obtained with an I bar 8 cm. (3 5/32 inches) high, and capable, therefore, of resisting, with the ordinary guarantees of safety ($\sigma = 12$ kgr. per cm² [$\sigma = 171$ lb.per square inch]), a bending moment of flexure of about 24 000 kgr.-cm. (20 830 inch-pounds).

These dimensions for the connecting girder are also justified by a theoretical investigation into the composite sleeper, and give a fairly close approximation to the characteristics of the timber sleeper.

Taking as basis the maximum bending

moment, *i. e.* 240 Kgr.-M. (1 736 foot-pounds), we have prepared for the composite sleeper the elasticity curve corresponding to the units of live load already adopted for the timber sleeper and the equi-strength girder sleeper (§ 7); this curve is given in figure 30 *b*).

It will be seen that the elasticity curve for the composite sleeper is very close to that for the timber sleeper, although the first consists of a single arc, whereas the second is formed of two arcs of very different radius ($\rho =$ about $3.4\ \rho'$).

Thus, under the highest constant moments that can be admitted with safety to the material, the timber sleeper and the concrete sleeper have very similar elastic deflections, and, therefore, other conditions being equal, the maximum values attained by the bending moments under particular stress conditions will be much the same in the two types of sleeper.

Having stated this fact, we would remark that the most unfavourable theoretical case of accidental stress for the girder sleeper is that shown in figure 28 *b*), where the corresponding maximum bending moment is in general equal to 0.75 P. On the other hand, this case cannot arise with the composite sleeper, in which the reaction which may affect the middle part is negligible, not only because of the very limited horizontal surface of the girder, but also because of its considerable superelevation above the supporting surface of the blocks (tamping level). The most unfavourable case in theory will, therefore, be that of figure 28 *a*), which, with a length of block of 70 cm. (2 ft. 3 1/2 in.), will give rise to a maximum bending moment of 0.35 P. We have seen (§ 7) that these moments do not occur in practice; nevertheless, making due allowance for what has been said in the preceding paragraph, it may be taken approximately that for the timber sleeper and the composite sleeper there are equal guarantees of safety whenever the theoretical maximum bending moments and the minimum moments of resistance bear the same ratio to each other.

The moment of resistance of the connecting girder should, therefore, be equal to

$$\frac{80\,000}{75} \times 35 = \text{about } 37\,000 \text{ kgr -cm. } (32\,100 \text{ inch-pounds})$$

i. e. about 50 % higher than the effective moment of 24 000 kgr.-cm. (20 830 inch-pounds).

At this point we should recall what has been said in § 3. As it is a question of determining the cross section of reinforced concrete which shall have the same strength as that of the timber sleeper, we allowed in the former the excessive stress of 56.3 kgr. per cm^2 (745 lb. per square inch), considering that the cross section of the timber sleeper (24 × 14 [9 7/16 × 5 1/2 inches]) was not always present in practice and was capable of appreciable reduction, particularly in the middle section. Consequently, if, for the purpose of comparing the two types, the two cross sections contemplated have been considered as equi-strong, the moment of safety of about 80 000 kgr.-cm. (69 400 inch-pounds) which has been attributed to them is none the less higher than that which can be safely admitted (1).

As regards the connecting girder, there is no reason to fear either reductions of cross-section or insufficient strength, and therefore a stress of 12 kgr. per mm² (17 000 lb. per square inch) is admissible in practice.

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(1) This fact does not at all modify the conclusions arrived at up to the present, in which the moment of safety has not been taken into consideration, with the exception of the determination of the elasticity curves in figure 30; but in this respect a diminution in the value of the moment of safety of the timber sleeper and concrete sleeper can only still further corroborate what we have put forward.

If, for example, it were desired to limit the unitary tension in concrete to 40 kgr. per cm² (568 lb. per square inch), the relative moment of safety would become about 56 000 kgr.-cm. (48 600 inch-pounds), and the moment of safety of the connecting girder

$$56\,000 \times \frac{35}{75} = \text{about } 26\,000 \text{ kgr.-cm. } (22\,600 \text{ inch-pounds}).$$

In view of what has been said it may be concluded that the capacity of the composite sleeper (of the dimensions indicated) to resist the most unfavourable accidental conditions of live load is quite comparable with that of the timber sleeper.

Before concluding our examination of the composite sleeper, let us consider one particular case of stress which is characteristic in the case of this sleeper.

In our inquiry we have adopted two limits for the coefficient of ballast, *viz.* C = 4 and C = 10. Let us now suppose that the lack of uniformity of the ballast and formation is such that, if we consider a single mixed sleeper, we have C = 4 under one block and C = 10 under the other. Let us see how, even under these conditions, which are of a quite exceptional character, the unitary stresses in the connecting girder assume values which are still permissible (¹).

Supposing, then, that for one block C = 4, and applying the formulæ (14′), we have :

$$\frac{P}{G} = 0.47; \qquad D = 8\,800;$$

$$y' = \frac{P}{D} = \frac{0.47}{8\,800}\,G$$

and for the other block, with C = 10, we have :

$$\frac{P}{G} = 0.60; \qquad D = 21\,300;$$

$$y'' = \frac{P}{D} = \frac{0.6}{21\,300}\,G.$$

If we now take an axle load of 16 tons, *i. e.* where G = 8 tons, we shall have :

$$y' = \frac{0.47 \times 8\,000}{8\,000} = 0.42 \text{ cm.}$$

$$y'' = \frac{0.60 \times 8\,000}{21\,300} = 0.23 \text{ cm.}$$

$$y = y' - y'' = 0.19 \text{ cm.}$$

The maximum moment of flexure will occur at the point of anchorage, and will be :

$$M = \frac{6\,EIy}{L^2}$$

where

$$E = 2\,000\,000;$$

$$\text{I for the girder INP80} = 77.8; \qquad L = 80.$$

$$M = \frac{6 \times 2\,000\,000 \times 77.8 \times 0.19}{6\,400} = 27\,800 \text{ kgr. per cm}^2 \text{ (395 400 lb. per square inch)},$$

and as the moment of resistance of the girder is W = 19.6 cm³.

$$\sigma = \frac{27\,800}{19.6} = 14.2 \text{ kgr. per cm}^2 \text{ (202 lb. per square inch).}$$

(¹) R. Desprets : « Note on some recent types of reinforced concrete sleeper » (*Bulletin of the Railway Congress*, July 1922.)

This tension is somewhat high, but is admissible in view of the exceptional character of the hypothesis adopted.

9. *Comparison of the two types of sleeper.* — To summarise what has been said above, it may be stated that :

a) In the conditions of stress derived from the Winkler hypothesis, the bending moments which occur in the timber sleeper are appreciably lower than those which occur in the reinforced concrete sleeper of equal strength, and the difference may amount to as much as 60 %. If, on the other hand, we take a block sleeper which has the same cross section as the preceding sleeper, its bending moments are lower than those of the corresponding timber sleeper for lengths of block not exceeding 70 cm. (2 ft. 3 1/2 in.).

b) In exceptional cases of live load, the moments which occur in the equi-strength girder sleeper also appear greater than the corresponding moments in the timber sleeper, although it is not easy to determine the extent of the difference. As regards the composite sleeper, certain of the accidental conditions, and precisely those which are the most unfavourable, cannot be verified : in any case, the length of the sleeper being fixed at 2.20 m. (7 ft. 2 5/8 in.), and the dimensions of the connecting girder being those stated above, the ratio between the ascertainable maximum theoretical moments and the minimum moments of safety of the timber sleeper and block sleeper is very similar. In other words, the block sleeper under consideration gives ample guarantees of safety, whether from the point of view of the strength of the materials or as regards the maintenance of the track level, the cant of the rails, the adjustment of the ballast, etc. Further, the elasticity curve for maximum stresses is very similar to that for the timber sleeper, which is a guarantee that the possible accidental stresses cannot assume, in the case of the mixed sleeper, very much higher values than those which occur with the timber sleeper and which have been shown by experience to be reasonable.

The composite sleeper is, therefore, capable of resisting the accidental stresses which are incidental to its structure.

Mention may be made of the following further points in favour of the block sleeper :

c) Even with a fairly wide block, such as that we have had in mind, it is possible to effect the tamping without difficulty, as the block is also accessible from the inside of the track.

d) The risk of the opening out of the track, which is a serious disadvantage with the reinforced concrete girder sleeper, is completely eliminated. The inner vertical surfaces of the blocks are held perfectly in position by the ballast, which ensures still greater transverse stability than in the case of the timber sleeper.

e) As there is no need of any elasticity in the isolated block, it is possible to increase its height, which gives a more satisfactory reinforcement.

f) The possibility of inversion of the bending moments being very limited, use may be made of a more economic reinforcement and, if desired, the block may be directly given the form of a solid of equal resistance.

g) Nothwithstanding the presence of a metal girder, the composite sleeper is more economic than the equi-strength girder sleeper. We have seen that a girder sleeper of uniform cross section, and 2.20 m. (7 ft. 2 5/8 in.) long, possesses very little strength because of the high values assumed by the bending moments in the middle section. In order to make an exact comparison, let us suppose that it is given the same length as the composite sleeper. Such a sleeper (leaving out of account the fastenings) would require 2.20 m. × 20 = 44 m. (144 ft. 4 in.) steel bar of 10 mm. (3/8 inch) in diameter, equivalent to 26.8 kgr. (57.7 lb.) of steel. In this, account is not taken of the possibility of reducing the compression reinforcement; this reduction is not, however, contemplated for the composite sleeper.

If we take 200 lire as the price of good concrete, the mixture of which is not less than 300 kgr. (19 lb. per cubic foot), and 160 lire as the price of 100 kgr. of round steel, the cost of a girder sleeper, excluding the fastenings, is as follows :

Steel : 26.8 kgr. at L. 1.60 . . . . = L. 43.00
Concrete : 0.14 × 0.16 × 2.20 = 0.105 m^3 at L. 200 . . . = L. 21.00
Total . . . L. 64.00

For the block sleeper, with a length of block of 70 cm. (2 ft. 3 1/2 in.), the weight of round steel (assuming the cross section to be uniform throughout) will be 17.1 kgr. (37.7 lb.). The volume of the two blocks will be 0.067 m^3 (409 cubic inches). We must also take into account the connecting bar which, to give a perfect connection, must be 1.70 m. (5 ft. 7 in.) long, and which we shall take at the price of 130 lire per 100 kgr. We then have :

Steel bar in the blocks : 17.1 kgr. at L. 1.60 . . . . = L. 27.40
Connecting bar : 10.2 kgr. at L. 1.30 . . . . = L. 13.25
Concrete : 0.067 m^3 at L. 200 . . . . = L. 13.40
Total . . . L. 54.05

The composite sleeper, having the same cross section and the same length as the girder sleeper, and offering, therefore, much better guarantees of safety, would cost about 85 % as much. It will, of course, be understood that these prices are only tentative. Passing from the theoretical to the practical dimensions, an appreciable reduction can be made, in accordance with what has been said in *e)* and *f)* above, in the quantity of steel bar used in the block sleeper.

10. — The theoretical conclusions which we have formulated above are entirely confirmed by practical results. With a composite sleeper of suitable design and construction, excellent results have in fact been obtained. We shall deal at greater length, in Part III, with this sleeper and the results obtained.

Before concluding this part of our study, we would repeat that if the hypotheses which we have adopted as our basis are more than sufficient for the object which we had in view, an exact determination of the stresses to which the permanent way in general and the sleepers in particular are subjected can only be effected by means of a long series of practical tests, and this is particularly the case as regards stresses on curves, at rail joints, etc.

Various attempts have already been made in this direction, though not perhaps on a sufficiently extensive scale or with the breadth of view that would be desirable. Among the most important investigations mention may be made of those of Coüard, Freund and Cüenot (1), the latter being of particular interest to us in that they relate to a type of composite sleeper of steel and wood (Devaux, Michel and Richaud sleepers). We give in figure 31 two average diagrams showing the deformation in composite sleepers and in timber sleepers. They appear to bear out the Winkler hypothesis : the timber sleeper has the characteristic elasticity curve, whereas the composite sleeper shows practically no deflection. It should be noted, however, that, for an equal width, the average deflection of the timber sleeper is greater than that of the two heads of the composite sleeper.

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(1) Cüenot : *Étude sur les déformations des voies des chemins de fer*, Bourg, 1905.

M. Cüenot's explanation of this fact is somewhat obscure; in our opinion the anomaly can only be attributed to a difference in the coefficient of ballast in the two test sections.

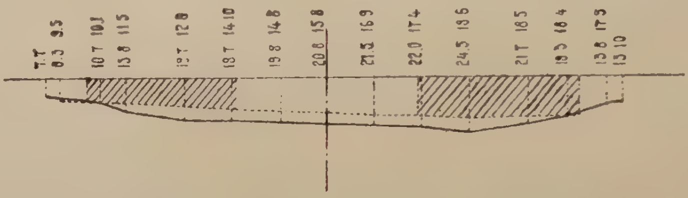

Fig. 31.

Finally, we learn that for some time past certain American railways have been carrying out a series of careful investigations and experimental tests in regard to the permanent way itself, on the large scale characteristic of such tests in the United States; we do not, however, yet know the results of these investigations.

PART III.

1. — We have seen in Part II that the composite sleeper affords a good theoretical solution of the problem of the reinforced concrete sleeper, and that a suitably designed type has real chances of success, or at any rate more so than the uniform girder sleeper.

As we mentioned in Part I, the most characteristic type of composite sleeper, and perhaps the first to be adopted, is the Vagneux type, from which the S. C. A. F. T. sleeper has been derived. The Vagneux type is much used on important French railways; the second sleeper is about to be introduced on a large scale on the North of Milan Railways.

We shall give a concise description of the first, and we shall refer to the results obtained with it in practice; we shall then describe an example of the second type (F. N. M. I. sleeper), now being made for the North of Milan Railways. In referring to this second type, we shall mention the results obtained in laboratory tests.

Finally, we think it will not be without interest if we describe the methods of construction of the F. N. M. I. sleepers, and the workshop where they are produced.

2. *The Vagneux composite sleepers.* — Figure 32 shows the type of Vagneux sleeper in use on the Paris-Lyons and Mediterranean, on the Northern Railway of France, etc.

It will be noticed that the block has a shape recalling that of a solid of equal resistance to bending. The reinforcement of the block consists of 4 bars of 9 mm. (0.354 inch) diameter and 4 bars of 10 mm. (0.394 inch) diameter for the

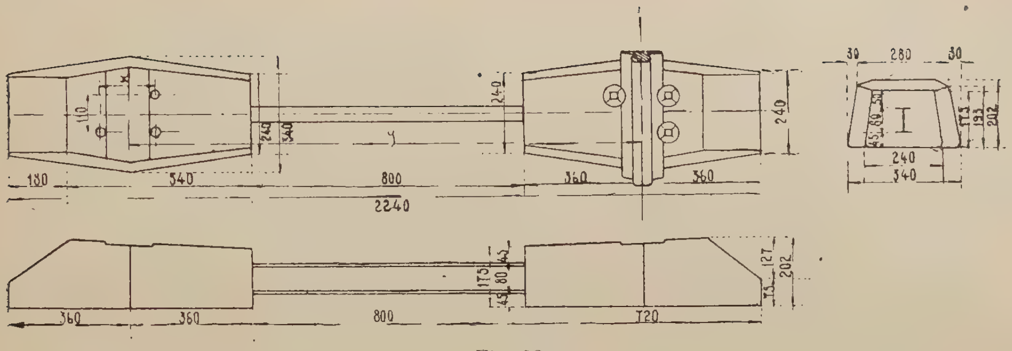

Fig. 32.

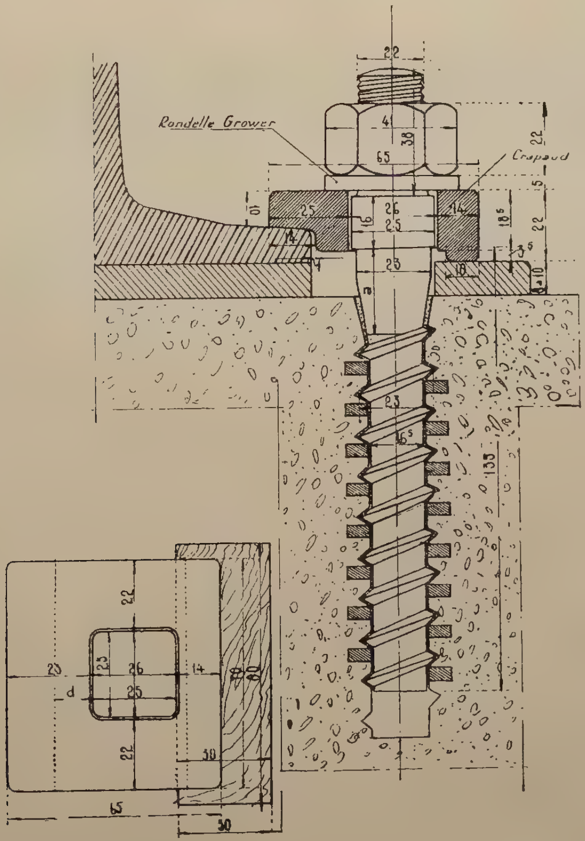

Fig. 33.

Explanation of French terms : Rondelle Grower = Grower washer. — Crapaud = Clip.

lower surface; the upper reinforcement consists of 2 bars of 9 mm. (0.354 inch) diameter. The maximum width of the block is 34 cm. (13 3/8 inches), its length is 72 cm. (2 ft. 4 3/8 in.) and its average height is 193 mm. (7 9/16 inches). The reinforcement forms what is known as a « basket », the construction of which we shall see more clearly when considering the F. N. M. I. sleeper.

It will be seen that the cross section of the block appears similar to that of the equi-strength girder. For the sake of economy, however, the reinforcement has been reduced (in particular for the upper surface) by increasing the height of the block, thereby using the materials to better advantage. This has been possible because the block need not have any elasticity, and, therefore, it is permissible to increase the height of the block, of course within the limits imposed by service conditions.

The connection between the two blocks consists of the ordinary double-T type of bar, 80 mm. (3 5/32 inches) in depth, which is embedded deeply in the concrete block so as to ensure perfect solidity.

It will be remarked that the upper surface of the block is cast at an inclination of 1 : 20, in view of the fact that the Vagneux sleepers are used in France without the addition of a bearing plate.

We may note in this connection that the metal bearing plate is not used by several of the large French railway companies, not only in the case of reinforced concrete sleepers, but also in the case of ordinary timber sleepers.

A recess is formed at the rail seat, in which is placed the resilient packing, which is generally made of wood.

The rail is generally fastened to the block by means of a Thiollier fitting, embedded in the concrete, on the system already described in Part I (fig. 16). Here, however, the method is somewhat varied, in that greater play is left between the spiral and the coach screw, as well as between the concrete and the coach screw : the gap is then filled with a special quick-setting bitumen. For this purpose, when laying the sleepers in the track, the coach screws are dipped into molten pitch before being screwed home.

In order to avoid the twisting of the coach screws which might result from the absence of the metal bearing plate saddle, use is often made of tempered steel coach screws.

Another system of fastening, or rather a variant of the first system, also employed with the Vagneux sleepers, is that in which use is made of the coach screw clip. As will be seen from figure 33, the coach screw is threaded at the top to take a regulating nut. Between the nut, the rail and the sleeper a clip is inserted which is in many respects similar to the clip which is in use on the Italian railways for the 46 kgr. (92.74 lb. per yard) track, with the definite object of preventing the coach screw from being damaged.

The resilient packing between the rail and the sleeper is usually made of compressed wood (elm) or teak, either impregnated or unimpregnated.

A « joint support » (see fig. 34) is often placed under the rail joint designed as a composite sleeper giving a supported joint, with the advantages and disadvantages with which we are familiar.

3. *Practical results obtained with composite sleepers.* — The type of sleeper described, as well as other types of a similar character, have as we have said, been introduced on a large scale on various French railways. In view of the satisfactory results obtained, the Administration of the North of Milan Railways, being desirous of arriving at a solution of the question of reinforced concrete sleepers, instructed two of its officials (¹) to inspect the composite

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(¹) M. Albricci, the Permanent Way Engineer, and the writer of the present article.

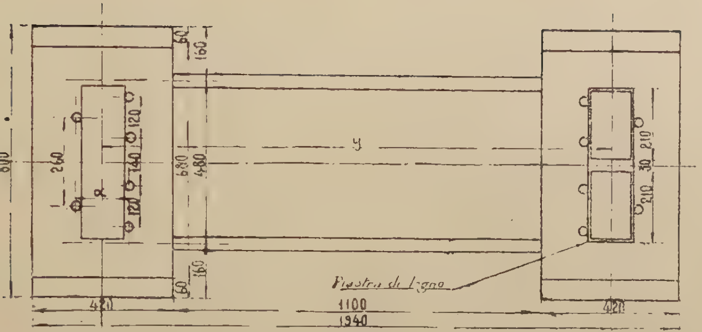

Fig. 34.

Explanation of Italian terms : Piastra di legno = Wood cushion.

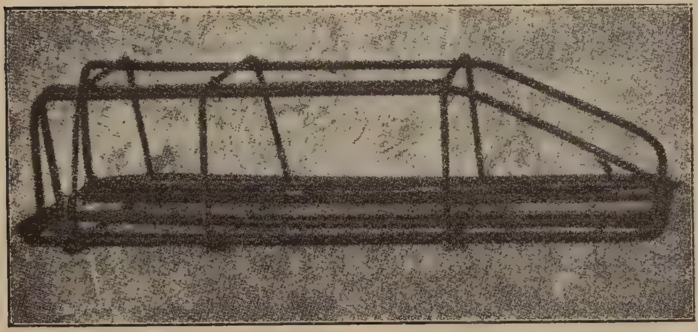

Fig. 35.

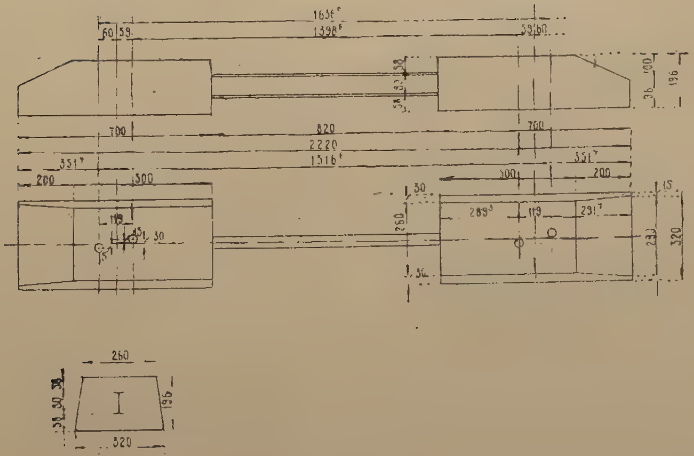

Fig. 36.

sleepers in use on the spot, to collect all available data and to make a detailed report.

At this point we think it advisable to remark that, as is well known to those who have had to deal with these matters, theoretical study, laboratory tests, or practical experiments on a limited scale, do not enable one to form a sound judgment with regard to the behaviour of the reinforced concrete sleeper in service.

Calculations, although they may be of great service in determining the dimensions and proportions of a sleeper, obviously cannot take into occount the accidental and secondary stresses which, by continual repetition, may have quite unexpected consequences. The problem of the reinforced concrete sleeper is essentially a railway problem, rather than a problem of construction. Laboratory tests cannot give entirely conclusive results in view of the impossibility of reproducing actual service conditions in the laboratory. In the same way, pracical experiments conducted on a small scale cannot be decisive. As we have already explained, sleepers laid in small numbers, even though on main lines, are not placed under normal conditions. In the first place, the short length of the section in question does not ensure that the most unfavourable service conditions will be encountered, unexpected conditions depending either on the profile and lay-out of the line (gradients, curves, etc.), or on the characteristics of the formation (more or less clayey soil, bottom ballast, embankments, cuttings, etc.), or again on the nature of the ballast (gravel, sand, broken stone, etc.), or, finally, on the characteristics of the traffic (frequency and weight of trains, braking points, accelerating points, etc.). Further, when it is a question of only a limited number of sleepers, the supervision they receive is obviously more thorough and the upkeep more careful. The breaking of a sleeper at the rail seat may be due in many cases to a coach screw remaining loose some time, or breakage in the middle section may be due to defective tamping of the sleeper ends. These defects of maintenance normally affect a great number of sleepers, and if reinforced concrete sleepers are to be a success they must be capable of withstanding such conditions.

It would seem, therefore, that no great importance can be attached to the results of anything but experiments effected on a large scale; but these experiments, which are of course costly, can only be carried out in rare cases, and only then by large railways, and they must be followed with the greatest attention with a view to deriving all possible information therefrom.

The experiments carried out with composite sleepers in France are precisely of this nature. We need only mention that the Paris-Lyons-Mediterranean Railway alone has about 120 000 composite sleepers, either in use or under construction, that the Northern Railway has about 100 000, while the Eastern and State Railways, although not having so many, still have a very considerable number, not to mention the innumerable sleepers of this type in use on secondary, local and colonial (Algerian) railways.

As part of these sleepers had been in use since 1923, *i. e.* for a period of about three years prior to our visit, a careful examination of their condition was obviously very instructive. The French railway companies, and in particular the Paris-Lyons-Mediterranean and Northern companies, gave us every facility for carrying out the inspection, and placed at our disposal technical experts and staff, who accompanied us in our visits to the various centres and assisted us in carrying out a most minute examination of the sleepers in use.

The sleepers examined were completely cleared of ballast and even, in certain cases, taken right out of the track for purposes of examination for fissures or deterioration of minor importance. Great care was taken in the examination of the sleeper fastenings, and most particular attention was paid to the attachment of the connecting bar to the blocks, where the formation of fissures or the disintegration of the concrete seemed most likely to occur.

Confining ourselves for the moment to noting only a few of the observations made, we will mention, as an example, that out of about 700 sleepers examined near to Plombières-lez-Dijon Station on the Paris-Dijon line, only five showed signs of deterioration. One of these five showed a small fissure at the point of connection with the bar, which, however, was found to be purely superficial; three showed superficial chipping in the upper section of the block, due probably to the diameter of the coach screw being slightly greater than that of the hole, which had been bored in advance; in the fifth and last sleeper, the edges were worn down on one side of one of the blocks. The line between Paris and Dijon is, of course, one of the busiet lines in Europe : each track is used by about 70 trains a day, at speeds exceeding 100 km. (62 miles) an hour. The sleepers examined were laid both on the straight and on curves; the ballast generally consisted of large broken stone, for the most part without sand. The sleepers had been in position since 1923, and none of them had been changed.

On the line from Paris to Brussels (Northern Railway), near to Chauny Station, composite sleepers were examined which had been laid in 1925. Out of about 400 sleepers examined under the conditions described above, only in two cases did one of the blocks show signs of deterioration. The first showed signs of disintegration of the concrete, but without fissures; and the other block showed fissures radiating from the connecting bar. The ballast consisted of very small, very hard and very unyielding broken stone. The track was used by about 80 trains a day, running at an average speed of 120 km. (75 miles) an hour, often, however, reaching 135 km. (84 miles). Side by side with the mixed sleepers there were in the track a certain number of concrete girder sleepers which had been laid at the same time : a large proportion of these showed deep fissures, and many of them, no longer fit for use, had been withdrawn from the track and were lying by the side of the line : they showed the characteristic fissures and disintegration at the middle and at the supports.

Other composite sleepers, also on main lines, were examined near to Dijon Station and also showed satisfactory results, although here maintenance work had been somewhat neglected at several spots, and the ballast consisted of quarry gravel with a large admixture of sand and, in some places, contamined with clay.

The examination of numerous other sleepers on tracks and lines of lesser importance gave, as was to be expected, even better results than those described above.

We witnessed the actual laying of sleepers, and were careful to visit the centres where the sleepers are constructed, in order to ascertain whether the good results observed might not be attributable to particular processes of construction. The workshops organisation and the materials used appeared to us to be very good, but we did not notice any special processes which could not be readily carried out in Italian workshops.

4. *The F. N. M. I. sleeper.* — In view of the results referred to in the preceding section, the North of Milan Railway Company decided to design a type of composite sleeper for use on its lines, and intrusted its construction to the « Società Cementi Armati Ferro Tramviari » (S. C. A. F. T.).

The F. N. M. I. sleeper is shewn in the

diagrams and photographs above, and is in many respects similar to the Vagneux mixed sleepers, except for several characteristics to which reference will be made later. It was designed for axle loads of 20 tons, without further allowance for dynamic stresses. Figure 36 gives a diagram showing the details of the sleeper, and a photograph of it is given in figure 37.

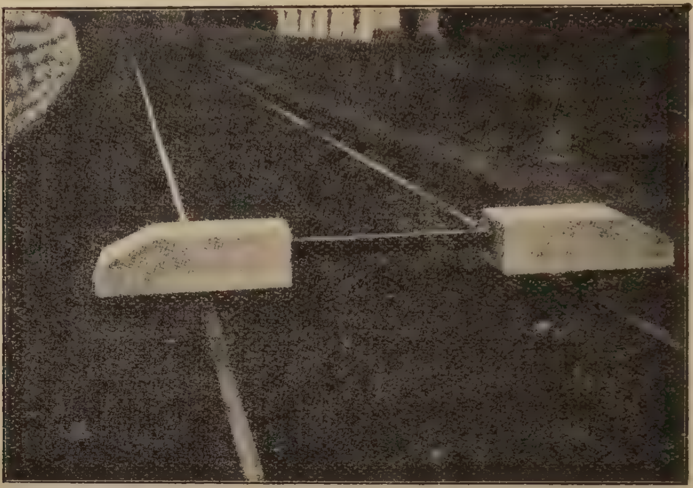

Fig. 37.

The reinforcement of the block is shewn in figure 35. It consists of a kind of basket framed of bars 9 and 10 mm. (0.354 and 0.394 inch) in diameter, bound together by transverse bars of 6 mm. (0.236 inch) diameter. This framework provides a lower reinforcement of four bars of 10 mm. and four of 9 mm. diameter, with a total cross section of 5.7 cm² (0.88 square inch), and an upper reinforcement of two bars of 9 mm., with a total cross section of about 1.3 cm² (0.20 square inch).

The block is 70 cm. (2 ft. 3 1/2 in.) long, 32 cm. (1 ft. 9 1/6 in.) wide at the base, and has a maximum height of 196 mm. (7 11/16 inches). Except for the outer chamfered edge, the lower and upper surfaces of the blocks are parallel, as it was decided to retain the inclined bearing plates under the rails in order to obviate deformation of the coach srews.

Connection between the blocks is afforded by the above-mentioned double-T bar, 80 mm. (3 5/32 inches) in height, embedded for a length of 43 cm. (16 15/16 inches) in the blocks, with a free space between the blocks of 82 cm. (2 ft. 8 1/4 in.).

Attachment is by means of the ordinary Thiollier fitting, which is embedded in the concrete; but in this case use is not made of bitumen, and, consequently, the play between the spiral, coach screw and concrete has been reduced to a minimum.

As has been stated above, the sleeper has been designed for axle loads of 20 tons, and the calculations have been made by applying the methods to which detailed reference has been made in Part II, and by considering the block as rigid, that is to say, neglecting the influence exercised by the elastic deflections on the distribution of the reactions. In these calculations the coefficient of ballast has been taken as 4 and 10. Although this is not the case in practice, the sleeper has been regarded as isolated and, therefore, the co-operation of adjacent sleepers has been left out of account. This case must naturally be regarded as allowing for the extreme limits of stress and, consequently, somewhat high unitary stresses are quite admissible. The composite sleeper has also been compared with the ordinary timber sleeper in accordance with the criteria described in section 8 of Part II, and the unitary stress for the connecting bar has been determined so as to correspond to stress conditions equivalent to 65 kgr. per cm^2 (924 lb. per square inch) in the timber sleeper.

The main results of the calculations are as follows :

I. — *Winkler hypothesis Zimmermann and Hoffmann formulae.*

$C = 4.$

Maximum stress in the concrete
σb = about 33 kgr. per cm^2. (469 lb. per square inch)
— — in the steel in tension
σf = about 526 kgr. — (7 480 lb. — —).
— — in the steel in compression
$\sigma' f$ = about 175 kgr. — (2 489 lb. — —).

$C = 10.$

Maximum stress in the concrete
σb = about 34 kgr. per cm^2. (484 lb. per square inch).
— — in the steel in tension
σf = about 544 kgr. — (7 740 lb. — —).
— — in the steel in compression
$\sigma' f$ = about 181 kgr. — (2 575 lb. — —).

II. — *Neglecting the influence of elastic deflections on the distribution of the reactions.*

$C = 4.$

Maximum stress in the concrete σb = 32.5 kgr. per cm^2. (462 lb. per square inch).
— — in the steel in tension
σf = 525 kgr. — (7 470 lb. — —).
— — in the steel in compression
$\sigma' f$ = 174 kgr. — (2 475 lb. — —).

$C = 10.$

Maximum stress in the concrete
σb = about 32 5 kgr. per cm^2. (462 lb. per square inch).
— — in the steel in tension
σf = about 525 kgr. — (7 470 lb. — —).
— — in the steel in compression
$\sigma' f$ = about 174 kgr. — (2 475 lb. — —).

III. — *Neglecting any support from adjacent sleepers, and under the same conditions as in the second case.*

Maximum stress in the concrete
σb = about 51 kgr per cm². (725 lb. per square inch).
— — in the steel in tension
σf = about 820 kgr. — (11 660 lb. — —).
— — in the steel in compression
$\sigma' f$ = about 275 kgr — (3 910 lb. — —).

IV. — *Shearing stress under the conditions of the second case.*

Maximum stress in the concrete . . σb = 2 kgr. per cm². (28 lb. per square inch).
— — in the steel . . . . σf = 800 kgr. — (11 380 lb. — —).

V. — *Bending stress in the connecting bar, due to exceptional live load conditions and equivalent to a bending stress in the timber sleeper of* 65 kgr. par cm². (924 lb. per square inch).

σf = about 1 200 kgr. per cm². (17 070 lb. per square inch).

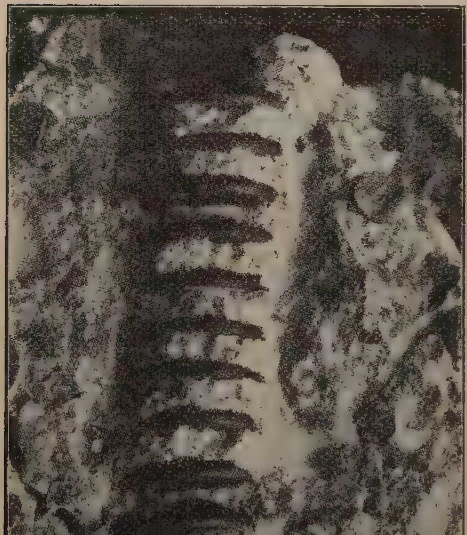

Fig. 38.

The mixture of the concrete is about 350 kgr. of cement per cubic metre (22 lb. per cubic foot) of mixture. The maximum size of gravel is 3 cm. (1 3/16 inches).

Great care was taken in applying the mixture round the metallic spiral, in order to ensure complete solidity. Figure 38 shows the appearance of the Thiollier spiral in the body of a sleeper which has been intentionally broken to pieces with hammer and chisel : the perfect cohesion between spiral and concrete is visible.

The volume of the two blocks, that is to say of the whole of the concrete, is 80 dm³ (2.82 cubic feet) the weight of the reinforcement in each frame is 4.5 kgr. (9.9 lb.), and therefore the total weight of the reinforcement is 9 kgr. (19.8 lb.). The weight of the connecting bar is 10 kgr. (22 lb.).

It will be seen that, compared with the equi-strength block sleeper, the F. N. M. I. sleeper, although designed to bear greater excess loads, is more economic as regards the quantity of steel used in its construction, because of the reduction in the upper section of the reinforcement. It is, in fact, perfectly logical to reduce the upper reinforcement, in view of the fact that the inversion of moments is very limited, if indeed it is possible.

The upper part of the block might be subject to tensile stress, when there would occur a distribution of the reactions of the nature of that indicated by letter *a)* in figure 28 (Part II), that is to say, when, for one or both of the blocks, the resultant of the reactions found itself nearer to the middle of the sleeper than the point of application of the corresponding excess load (middle of the block). In this case, however, the connecting bar would also be subjected to the same bending moment, but this bar, in view of its elasticity, allows of adjustment of the block, and, therefore, the reduction or elimination of this moment. In any case, then, a heavier upper reinforcement is indicated as, even under actual conditions, the resistance of the block to positive moments is greater than that of the connecting bar, whereas the above-mentioned moments can never assume in the former higher values than in the latter.

Further, as regards shearing stresses, which are taken not only by the bent steel of the main reinforcement frame, but also by the transverse bands of the frame, the sleeper possesses good powers of resistance.

Figure 39 shows the normal cross section, in the track, of the composite sleeper on single and double track lines. It will be seen that the volume of the ballast, for an equal sleeper pitch, is appreciably less than for timber sleepers.

From the same figure it is also clearly evident that the track acquires a transverse stability superior not only to that obtainable with the uniform girder type of concrete sleeper, but also perhaps to that which can be given by the timber sleeper, the reason being the resistance exercised by the ballast itself in reacting against the vertical inner face of the blocks. Figure 41 represents a section of track laid with composite sleepers.

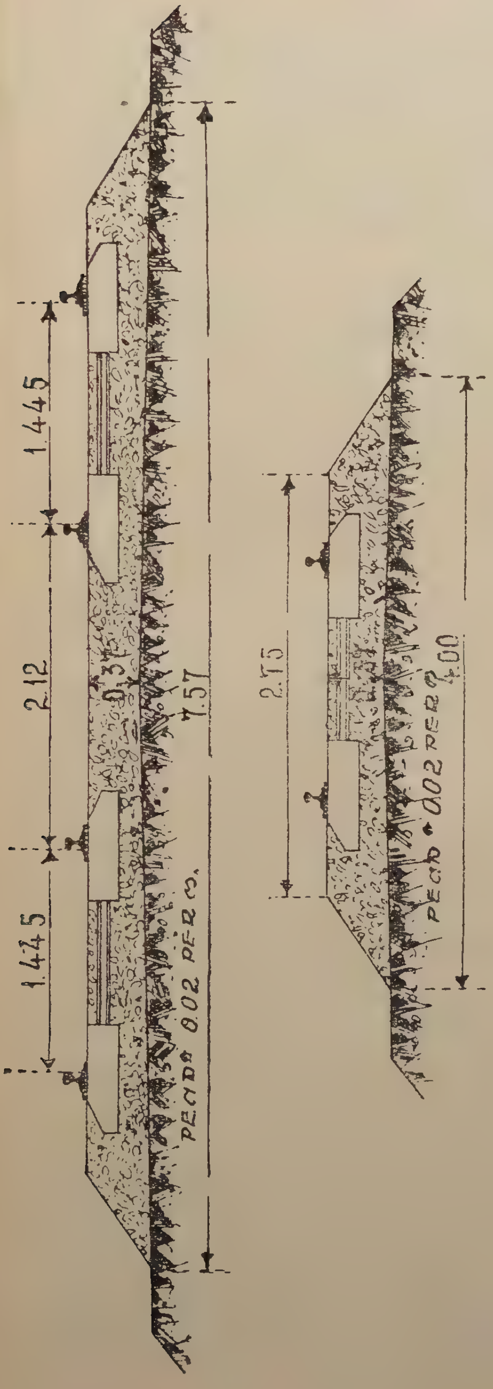

Fig. 39.

Explanation of Italian terms : Pend• 0.02 per m. = Inclination of 20 mm. per m. (1 in 50).

5. *Laboratory tests.* — The experimental tests with the F. N. M. I. sleeper were

carried out in the constructional materials testing laboratory of the Royal Polytechnic School, Milan, and related principally to :

a) Pulling out the coach screws,
b) Crushing the sleeper,
c) Pulling out the connecting bar,
d) Elasticity.

a) *Tests relating to the pulling out of coach screws.* — An ordinary permanent way coach screw was screwed home to the normal length, in such a manner as to leave a space of about 20 mm. (3/4 inch) between the underside of the coach screw head and the upper surface of the block.

The surface of the block was applied against a metal ring, having an internal diameter of 175 mm. (7 inches) and a thickness of 60 mm. (2 3/8 inches). This precluded the possibility of the results being affected in any way by the presence of the support.

In a first test, carried out with an outside coach screw, breakage of the coach screw (14 mm. [9/16 inch] diameter) occurred at a tension of 7 500 kgr. (16 530 lb.), at the third thread, and resulted in the destruction of the spiral and the displacement and breaking up of the concrete, but only in the part corresponding to the broken section of the coach screw. On examining the damage to the concrete it was discovered that the remaining portions of the spiral and coach screw had not moved, and that the surrounding concrete showed no signs of fissures.

In order to determine the effect which the pulling out of the coach screw had upon the structure of the concrete, the test was repeated under the same conditions upon the inner coach screw of the same block; the distance between the centres of the two screws was 120 mm. (4 3/4 inches). At a stress of 6 500 kgr. (14 330 lb.) the coach screw broke, at the fourth thread, accompanied by destruction of the spiral and breakage of the concrete around the broken part of the coach screw. It follows, therefore, that although the pulling out of the first coach screw may have caused some diminution in the resistance of the adjacent spiral, the effort necessary to tear out the latter from the concrete is always greater than the resistance of the coach screw shaft.

The third test confirmed the results of the preceding two. It ended with the breakage of the coach screw at a stress of 7 550 kgr. (16 660 lb.) and with identical results to the spiral and concrete.

b) *Tests with reference to the crushing of the sleeper.* — The block was placed and centred on a plate of 35 cm. (13 3/4 inches) edge. Application of the live load was made by means of the ordinary bearing plate on which were carried steel weights having the same width as the rail (100 mm. = 3 15/16 inches).

In a first test, under a load of 83 tons, the commencement of a slight fissure was noticed at the middle of the larger side of the block extending towards the point of application of the loads. As the load was increased, new longitudinal fissures appeared, until breakage occurred at a load of about 122 tons.

In a second test, carried out under identical conditions, the first slight fissures appeared under a load of about 60 tons, while breakage occurred under a load of 123 tons.

c) *Tests with reference to pulling out the connecting bar.* — A sleeper was cut into two parts by sawing through the middle of the connecting bar. The inner vertical face of the block was then placed on a ring of 175 mm. (7 inches) internal diameter and 60 mm. (2 3/8 inches) thick; then a pull was exerted on the sawn off end of the connecting bar.

At a tension of about 14 800 kgr. (32 600 lb.) breakage of the bar occurred near the gripping jaws of the testing machine, where the bar had been weakened by the sawing. There was not the slightest trace of the bar having slipped inside the block, and there were no fissures in the concrete.

In a second test made before the fastening had been strengthened the bar was torn completely out of the block at a stress of 24 000 kgr. (52 900 lb.), leaving no trace of any corresponding slipping or any fissures in the concrete.

b) *Elasticity tests.* — The sleeper was placed under the testing machine as shown in figure 40. Pressure (P) was applied twice up to 5 000 kgr. (11 000 lb.), then diminished gradually to zero in order to ascertain the re-adjustment required, neglecting the small permanent deformations. The deflections were then read on three fleximeters up to a stress of 5 500 kgr. (12 100 lb.), and then, on the middle apparatus only, until complete deformation of the bar.

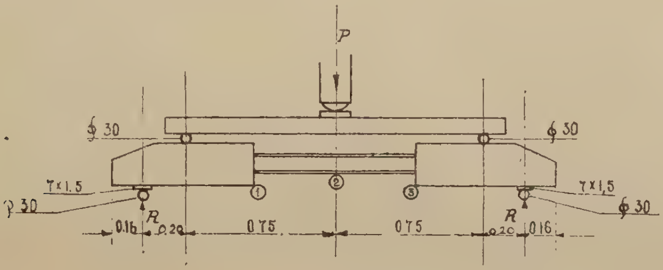

Fig. 40.

Complete deformation of the bar took place between tensions of 6 500 and 7 300 kgr. (14 300 and 17 000 lb.), corresponding to bending moments of 65 000 an 73 000 kgr.-cm. (56 420 and 63 360 inch-pounds) and, therefore, to a maximum unit stress in the bar of between (33 and 38 kgr. per mm² = 47 000 and 54 000 lb. per square inch).

Here again there was no trace of slipping of the bar in the concrete, nor of fissures in the concrete around the bar. There were naturally no fissures in the blocks themselves. We give below the deformation figures, measured in 100ths of a millimetre (in inches) :

Load (P.).		Moment.		1		2		3	
Kgr.	Pounds.	Kgr.-cm.	Inch-pounds.	1/100 mm.	Inches.	1/100 mm.	Inches.	1/100 mm.	Inches.
500	1 100	5 000	4 340	0	0	0	0	0	0
1 500	3 300	15 000	13 020	170	0.0673	220	0.0866	172	0.0677
2 500	5 500	25 000	21 700	339	0.1335	437	0.1720	340	0.1339
3 500	7 700	35 000	30 380	483	0.1902	619	0.2437	483	0.1902
4 500	9 900	45 000	39 060	616	0.2425	784	0.3087	612	0.2409
5 500	12 100	55 000	47 740	768	0.3024	984	0.3874	764	0.3008
6 500	14 300	65 000	56 420	...	...	1 226	0.4827	...	...
7 300	17 000	73 000	63 360	...	...	1 746	1.1700	..	...

As we have several times pointed out, the results mentioned in *a*), *b*) and *c*) would, in the case with which we are concerned, possess only a very limited significance unless they were confirmed by the results of experience with sleepers in use on the permanent may. They can, however, give an idea of the quality of the materials used and of the suitability of the relative proportions of the component parts of the sleeper

The tests with reference to the pulling out of the spiral, for example, while giving excellent results, are not in themselves sufficient to guarantee the good performance of this system of attachment in practice, as the continuous and uniform action of the testing machine is not to be compared with the quasi-vibratory action exercised by the foot of the rail and with the vibrations due to secondary stresses.

Similarly, the crushing test does not furnish a complete guarantee as regards the constitution of the concrete block. The test reproduces only in a slight degree the bending moments which occur in practice, whether because of the limited length of the bearing plate or because of its practically absolute rigidity.

The elasticity test is more instructive, and its results correspond closely with those indicated by calculation. The practical values are invariably lower than the theoretical values, probably because the values adopted for the units of elasticity are too low.

Up to the breaking-point limit the bar apparently remains absolutely fixed in the concrete, under stress conditions of the same nature as those to which it is subjected in practice, although, of course, of much greater intensity.

This test also supplies reassuring evidence as regards the risk of deformation of the bar, such deformation not making an appearance until the bending moments have reached the elastic limit.

6. *Methods of construction of the F. N. M. I. sleeper.* — It may not be without interest if we conclude our report on reinforced concrete sleepers by giving a few notes regarding the methods adopted in the construction of the F. N. M. I. sleeper.

These sleepers are made in a special shop in the buildings of the old station at Erba Incino on the North of Milan Railways, in the vicinity of a sand and gravel quarry.

a) *Materials.* — The sand and gravel are obtained from the neighbouring quarry, which is connected by rail with the shop. Although the materials extracted from the quarry are already very clean, they are carefully washed before use.

The cement normally used is Montandon cement from Ponte Chiasso. Each delivery of steel and cement is tested in the laboratories before use.

b) *Mixtures.* — These are still made by hand, and have a cement content of 350 kgr. per m^3 (21.8 lb. per cubic foot), The mixture is semi-fluid, and therefore great care has to be exercised in compressing it in the moulds.

As soon as the introduction of mechanical moulding (to which reference will be made later) is complete, it will be possible to reduce considerably the proportion of water in the mixture.

c) *Reinforcement.* — The reinforcements are prepared separately on special stands (see fig. 42). Each reinforcement frame consists of three principal steel rods, one of which forms part of the lower reinforcement (bottom of the frame), while the two others, suitably bent and placed above the first, complete the frame, which is then bound together by three transverse bands 6 mm. (1/4 inch) in diameter.

d) *Patterns.* — The moulds used for the concrete are of a new type. They are made entirely of thin steel plate, and are arranged to take two sleepers. The bottom of the mould is fixed on a stand, and has slots to take the special plates carrying the patterns of the coach screws and Thiollier fittings.

The sleepers are moulded upside down, and all the walls of the moulds, with the exception of one, can be reversed or removed, so as to facilitate the setting of the concrete and to make it easy to remove the sleeper.

The special plates referred to above are interchangeable for the different moulds, and are bored in advance with two, three, or four holes, according to the nature of the superstructure. These holes

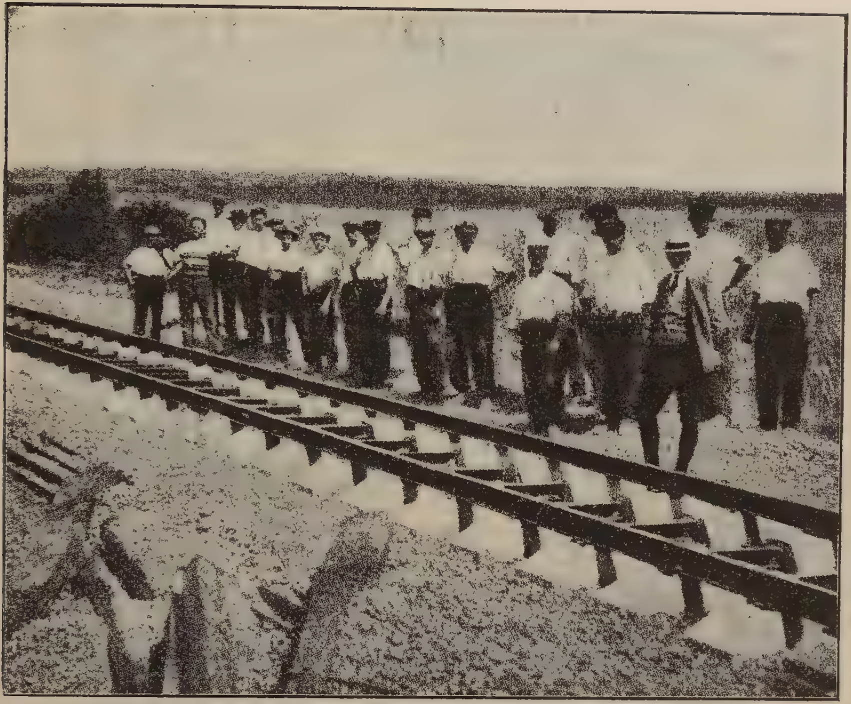

Fig. 41.

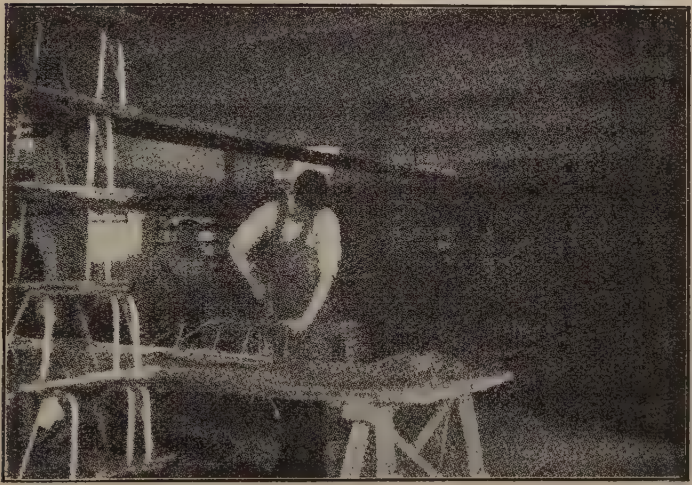

Fig. 42.

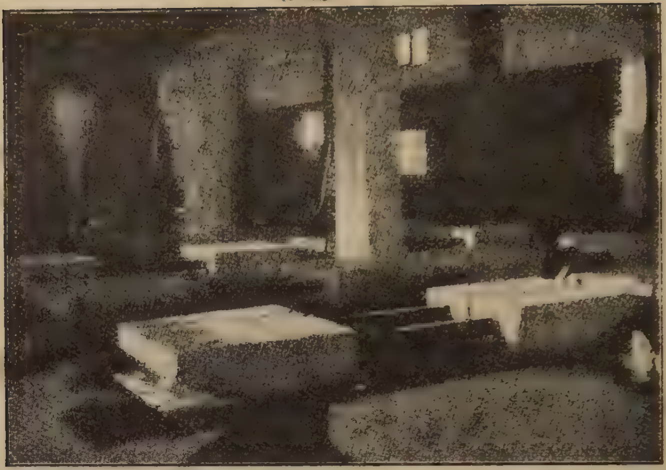

Fig. 43.

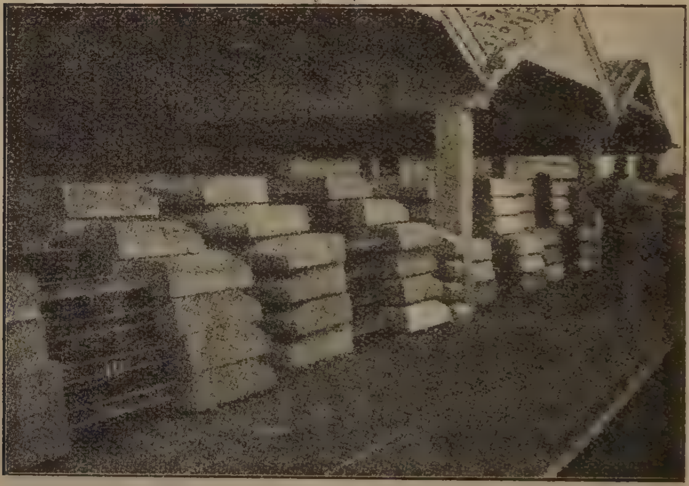

Fig. 44.

are spaced so as to allow, for widening of the gauge at curves, from 1.445 to 1.460 m. (4 ft. 8 15/16 in. to 4 ft. 9 15/32 in.), graduated in degrees of 2 mm. (5/64 inch).

The coach screw patterns have the same thread as the coach screws themselves, but slightly larger so as to facilitate the screwing up of the coach screws during the operation of laying on the track.

Figure 43 shows several sleepers lying

in the moulds after opening the walls of the moulds, and, on the right, several moulds ready to receive the reinforcement and concrete.

e) *Moulding.* — After having closed and greased the moulds, fixed and greased the coach screw patterns, screwed up on the latter the Thiollier spirals, and placed the reinforcement frames and connecting bar in position, the moulding is commenced and is completed within a few minutes.

In this connection it may be remarked that if there is one moulding operation in which, more than in any other, absolutely accurate compression of the mixture is essential, it is the moulding of reinforced concrete sleepers; and, therefore, in order to avoid the risk of negligence on the part of employees the use of mechanical apparatus is to be recommended.

For this reason a special apparatus has recently been designed and brought into use, which we shall describe as a « jarring table ». Its principal feature is a table formed of metal plates on which a double mould can be fixed. By means of eccentrics the apparatus is made to jar so as to cause the setting of the mixture. Numerous tests have been carried out which a view to determining the most suitable speed and degree of the vibrations, as, when a certain speed is exceeded, or when the rate of vibration is insufficient, the mass of concrete enters into a state of vibration without undergoing the desired binding action. Other tests were made to ensure that the vibrations do not cause a separation of the elements of the mixture. These tests were all entirely satisfactory.

It is certain that an apparatus of this kind not only means appreciable economy as regards the cost of labour, but gives a more satisfactory compression of the concrete, even in castings with complicated reinforcements, and also ensures that in this respect all the castings are practically identical; it therefore guarantees a uniform degree of reliability.

f) *Drawing out the patterns.* — On the day following the moulding the walls of the moulds are reversed and the coach screw patterns are withdrawn from the underside of the moulds. Two or three days after being moulded (according to the season) the sleeper is completely drawn out, prior to the painting of the connecting bar.

g) *Setting.* — One of the most interesting features of the process is the fact that the setting and hardening of the concrete take place under water. In the workshop there are large tanks in which the sleepers are piled immediately they are drawn out of the moulds. These tanks are then filled with water and the sleepers remain immersed for about twenty days.

[62. (01 & 669. 1]

The relative values of the tests employed during the inspection of steel for manufacturing purposes,

By Ch. FRÉMONT.

Fig. 1 to 10, pp. 170 to 173.

(*Génie Civil.*)

All specifications for steels for manufacturing purposes : machine parts, ship and boiler plates, girders, shafts, guns, axles, rails, etc., impose the tensile test as the principal test under the specious pretext that as the calculations are based on the tensile strength, the tensile test is of capital importance and is all sufficient.

Now the calculations of the strength of materials are based on the *static measurement* of the limit of elasticity of the metal, which limit must never be reached in service in the detail as designed.

Unfortunately the instantaneous stresses which may result from shocks or vibrations cannot be calculated : still less can we determine the interference points at which the maximum stresses occur.

Furthermore, as a rule the specifications in the case of steels, do not insist on the elastic limit being measured during the tensile test : they only specify the breaking load and the percentage elongation of the steel test pieces, as I pointed out in a pamphlet on the tensile testing of metals (1), wherein I went on to add :

« When, quite exceptionally, the specification requires the elastic limit to be measured, as there is no agreement as to its definition, the Steel works have taken advantage thereof, and are satisfied to note the beginning of the permanent deformation of the tensile test piece by measuring with compasses to the nearest tenth of a millimetre the distance apart of the gauge length pock marks.

« This method of measuring the elastic limit very much favours the Steel works, which can make it give almost any result they like, as I have already explained.

« On the other hand, the breaking strength required by all specifications is of no utility, as it does not satisfy any practical requirement and, as I have demonstrated, does not even enable the grade of metal to be classified.

« The second measurement required by all specifications, that of the ductility based upon the elongation of the gauge length of the test piece, is inaccurate, and this measurement, even if accurately determined, is without practical value, because a steel even when very ductile can be brittle and break under a light blow.

« Finally, in an article on the *fatigue* of metals, published in 1910 (1), I pointed out that metal parts always stood up well to static loads, as the failures occur-ring immediately after the parts are put into use, with deformations comparable with those of the tensile test pieces, are extremely rare, and that *the whole of the failures* are ascribable to dynamic stresses. The fracture is then, as a rule, the sudden ending of a more or less slow fissuration : it occurs without marked deformation, which at once distinguishes it from the fracture due to too great static loading. The usual static tensile test carried out by the conventional method therefore does not give any information as regards the strength the part will shew in use.

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(1) *L'étude de l'essai de traction des métaux*, (The tensile testing of metals) by Ch. Frémont. Experimental enquiries in industrial technology, 72nd pamphlet, 1927. Published by the author.

(1) See the *Génie Civil* of the 22 October 1910 (Vol. LVII, No. 25, p. 460).

« It is for this reason that as the tensile test of steels became general in the steel trade, engineers have frequently been deceived in their expectations, the many fractures in service of parts having satisfactorily passed the tensile test shewing that this method of inspection is valueless.

« To sum up, as the customary tensile test gives no information at all as to the strength of parts in service, it has not the value usually ascribed to it, and the inspection tests that ought to be required by the specifications should be the *etching* and *brittleness* tests.

The manufacturers being solely responsible for their products have therefore the greatest interest in knowing the practical value of the test methods they use in order to avoid the grave deception that results from using test methods that are anodine and afford no real guarantee as regards safety.

Welded Vessels. — In this connection, I have shewn (¹) that a locomotive boiler has successfully passed the regulation static hydraulic pressure test, whilst in certain parts the plate was cracked through four-fifths of its thickness (fig. 1).

This static hydraulic pressure test, whilst being useful to test the *tightness* of certain containers, such as steam boilers, reservoirs and bottles for compressed gases, etc., is in no way any guarantee for the public safety, and practising engineers know it.

In the same way, this test carried out to shew that in such containers the autogeneous weld is stronger than the original metal, as the « Société d'Encouragement pour l'industrie Nationale » has described in its Bulletin of July 1927 in the following terms, gives no information on the resilience of the material of the container :

The « Conseil d'Hygiène du Département de la Seine », being seized with the danger that might result from welded containers containing compressed liquid or dissolved gases, desired to obtain information as to strength of the welds of such vessels.

« Tests carried out before a Committee appointed by it on commercial bottles selected at random, as well as on containers made in its presence, were completely satisfactory.

« The bottles failed through the solid plate under hydraulic pressure continued until the bursting point was reached.

« Autogeneous welding can therefore be considered as giving absolutely perfect connections, the strength of which can be compared to that which would have been obtained if these same pieces had been entirely forged, extruded or pressed. »

Now this conclusion is absolutely erroneous. In fact, the static hydraulic pressure test does not give any information at all on the resilience of the material of the container tested, as we have noted above in recording the case of a locomotive boiler with a plate cracked through four-fifths of its thickness, and having, however, successfully passed the regulation hydraulic presssure tests.

In the same way, static tests carried out

(¹) See the *Génie Civil* of the 28 January 1928 (Vol. XCII, No. 4, p. 90).

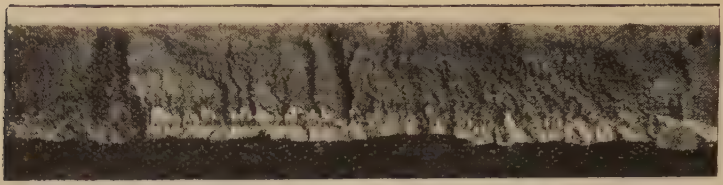

Fig. 1. — Section of a locomotive boiler plate cracked at the foundation ring level.

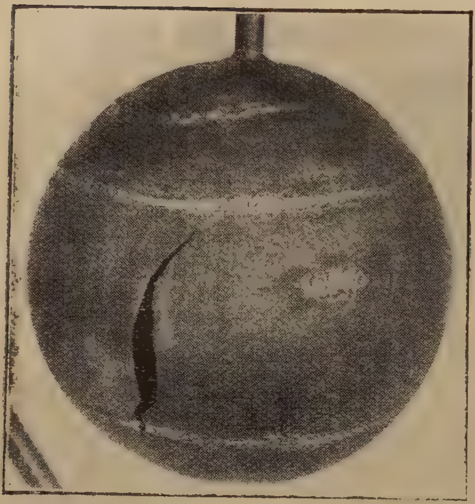

Fig. 2. — 9 7/8-inch diameter sphere made of steel plate 0.071 inch thick, welded autogeneously.

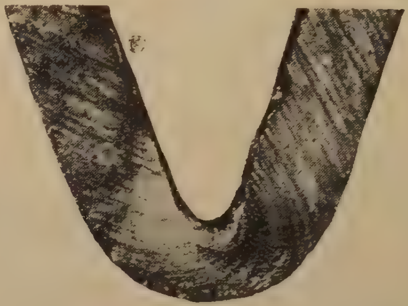

Fig. 3. — Unnotched test piece cut out of the solid metal of the sphere that did not break under the shock test.

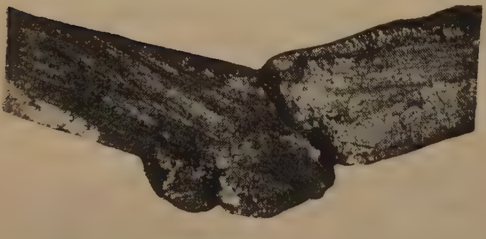

Fig. 4. — Unnotched test piece cut across the weld which separated under the shock test.

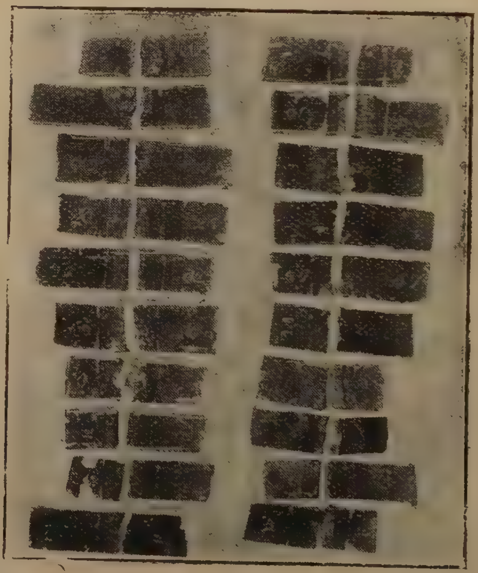

Fig. 5. — Series of twenty unnotched test pieces cut across autogeneous welds which broke under the shock test.

on welded test pieces do not any more give information as to their resilience : for example we obtained a good tensile test on a steel test piece brazed on a section at an oblique angle to the axis (1). It might be said that a test piece with a brazed joint has a higher static strength than that of the steel, since the steel failed and the brazing stood up, whereas all practising engineers know that a brazed joint is without resilience.

A test of the resilience of a metal can be made by shock. To test the resilience of a weld, it is necessary to cut a test piece which straddles the weld, that is to say, that the plane of the welded surface ought to be a prolongation of the plane along which the hammer strikes (2).

If the « Conseil d'Hygiène du Département de la Seine » had carried out on autogeneously welded containers, such shock tests, instead of the anodine hydraulic pressure test ,a result would have been obtained very different from that given in the *Bulletin de la Société d'Encouragement pour l'Industrie nationale.*

To demonstrate this, I took a sphere formed of pieces of steel plate welded together by autogeneous welding (fig. 2), which broke through the *solid metal* under a static hydraulic pressure test carried up to bursting point.

In the metal close to the fracture I took a small test piece, which I tested by a falling weight test : the test piece bent properly without shewing the slightest traces of cracking (fig. 3).

I then cut off a similar test piece, but this time across the autogeneous weld. Tested by the falling weight test, the test piece broke off sharply, the weld pulling away (fig. 4).

As it might have been supposed that it was due to a defect in the part of the weld tested, I had twenty test pieces cut out at various points, selected at random, across the welds of the sphere. Not a single one of the twenty test pieces stood the drop test (fig. 5).

This test shews that it will not do to let it be said that « autogeneous welding can give absolutely perfect joints, the strength of which can be equal to that which would result if the same parts had been entirely forged, drawn out or pressed ».

Not only have we no practical method of learning if a given weld has been satisfactorily carried out, but autogeneous welding, even when *perfectly done,* is always in the welded piece a source of weakness liable to cause the sudden failure, the possible cause of accident and even of disaster.

In reality, autogeneous welding is only used because of its convenience and its cheapness : it is therefore quite suitable for all cases in which the failure of the welded part will not lead to serious accident; but its resiliency being extremely low, it is not suitable for those cases in which fracture of the piece could have serious consequences.

It is of the greatest importance that practising engineers should remember that the falling weight test alone carried out in such manner that the weld lies on the same plane as that through which the weight falls gives any information as to the resiliency of the weld. The static tensile, torsion, bending, etc., tests, carried out on welded test pieces, like the hydraulic pressure tests of welded vessels, give results which are misleading.

Railway rolling stock axles. — To demonstrate the importance of the proper selection of the tests to be carried out when inspecting steel for constructional purposes, I will take as an example the test specified for steel axles for railway locomotives, tenders, carriages and wa-

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(1) See the *Génie Civil* of the 26 February 1910 (Vol. LVI, No. 17, p. 319) : « Experimental study of the resiliency of the welds. »

(2) See the *Génie Civil* of the 19 and 26 November 1910 (Vol. LVIII, Nos. 3 and 4, pp. 53 and 74).

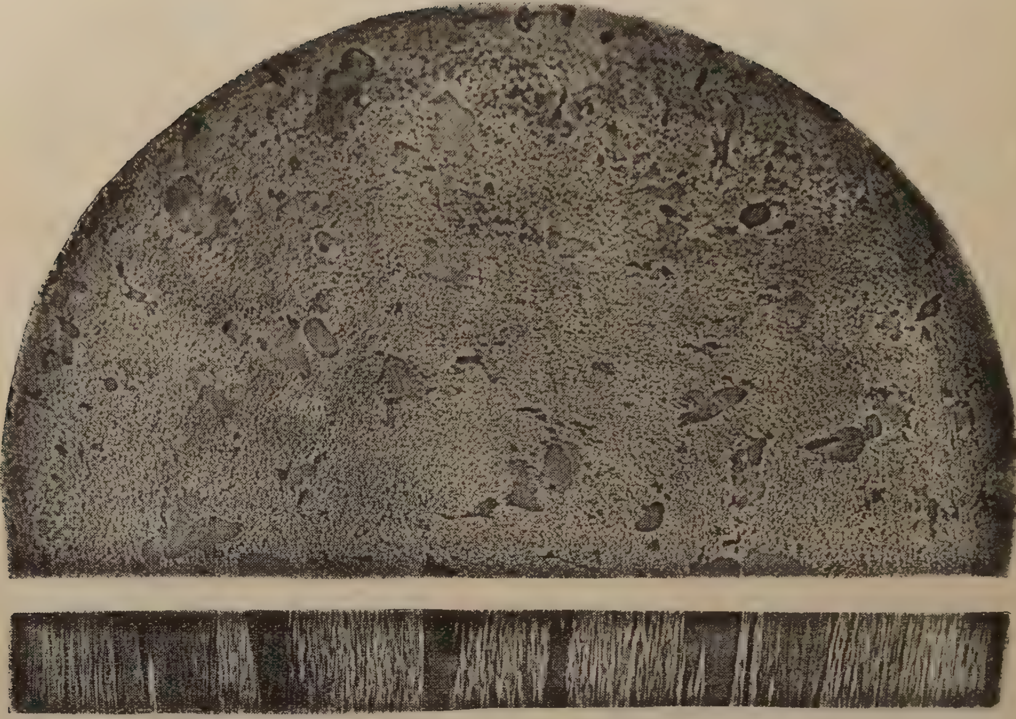

Fig. 6 and 7. — Macrographs of cross and longitudinal sections of a railway axle 5 1/8 inches in diameter.

The longitudinal dark veins correspond to segregation nodules in the steel visible on the edge of the cross section.

gons, because the breaking of an axle in service may result in a serious accident.

The Standard Specifications of the French Railways require the three following tests for such axles :

Drop test on the axle itself;

Tensile test;

Texture test.

I have shewn in my pamphlets Nos. 71 and 72 that the specified tests reveal no information as to the presence of segregation, the principal cause of the failure of axles. In order to study the harmful effects of such segregation, I took an axle which broke in service and cut from it a slice with which to carry out the tests of the metal.

After etching to ascertain the distribution of the segregated nodules, the slice was parted along a line selected so as to pass through the principal segregated points (fig. 6). On the rectilineal edge of this cross section can be seen the remaining part of each of the segregated nodules across which the longitudinal cut has been made, figure 7 being the macrograph thereof. On this latter figure will be seen segregated veins which exactly correspond to the portions of the segregated nodules visible on the cross section.

These segregated veins are well known

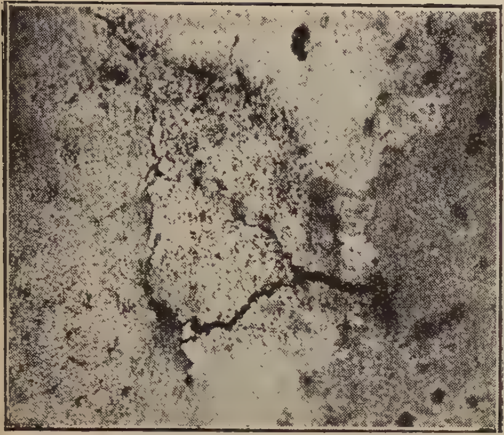

Fig. 10.

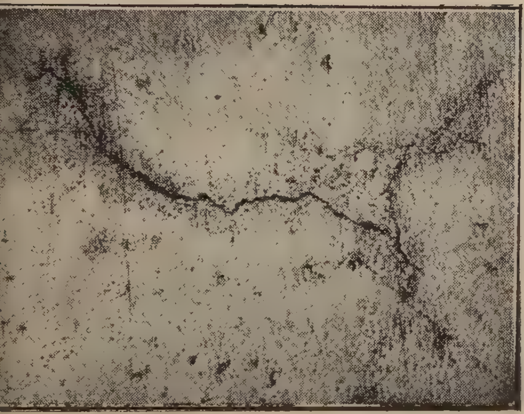

Fig. 9.

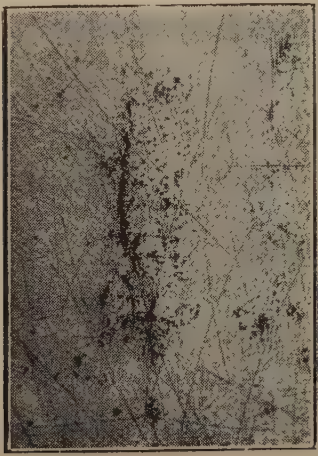

Fig. 8.

Figs. 8 to 10. — Beginning and growth of fissuration in the segregated nodules as the result of repeated blows.

to the Railway Inspectors, as when axles are being turned, they appear as *dark lines* following the generatrices of the cylinder turned. When the axle has been twisted in forging, the lines appear as helices. If the turned axle should oxydise even slightly, the dark lines rust at once. If a newly turned axle is etched, the dark lines then appear much more visibly, as shewn by figure 7.

These segregated nodules, revealed by the dark lines, are possible causes of failure of the axle, as we will demonstrate. If with a hand hammer the face of the slice which has not been etched is struck with successive blows gradually increasing in force, it will be noticed at a given moment that small fissures appear in the midst of the segregated nodules, and gradually grow as the hammering is continued (figs. 8, 9 and 10). The same phenomenon is noted with the axle in service : under shocks and vibrations the fissures begin and grow slowly in the midst of the segregated nodules until the section of the axle is sufficiently reduced by the fissuration to fail under a final blow.

The drop test on the body of the axle, laid down by the specification for the inspection of the axle, does not break the axle because the parts defective through segregation are supported and held together by the sound metal. In service the phenomenon is no longer the same because a creeping flaw develops, which inevitably ends in fracture after a shorter or longer period of service.

When carrying out an investigation into some rails, I pointed out (¹) that the internal fissuration of the rails was caused by the segregation of the steel, and that its beginning and growth depended upon the distribution and composition of this segregation. I then called

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(¹) 58th pamphlet : *Causes d'usure prématurée des rails* (Causes of premature wear of rails), p. 17. Paris, 1921.

attention to the fact that in steel ingots the piping varies in dimensions, in shape and even in position : it should therefore be expected that the same variations would be found in the contaminated part of the rails rolled from these ingots. The same thing will of cause take place in all parts manufactured from the ingot by forging or rolling : such as axles, shafts, etc.

As a result of this very thorough investigation into the quality of rail steel, the Italian State Railways, and also the Belgian, introduced into their Rail Specifications the *eliminating* test by macrographing. The *Génie Civil* of the 28 January 1928 gave extracts from these Specifications shewing the form in which the tests by etching is now specified in these two countries.

The Steel Makers have had to satisfy these new demands, and all visitors to the Metallurgical Section of the recent Exhibition of Raw Materials (¹) in Berlin admired the purity of the ingots exhibited. There is no doubt but that the French Steel Makers will follow the good example set and that we shall soon see a marked reduction, not only in the breakages of rails, but also of axles, shafts, etc.

(¹) A review of this exhibition which took place between the 22 October and the 30 November 1927, was given ~~in the~~ *Génie Civil* of the 11 Februari 1928 (Vol. XCII, No. 6, p. 142).

[62. (01 & 669 .1]

Strain effects in mild steel,

By Henry S. RAWDON,
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Figs. 1 to 15, pp. 177 to 187.

(*Engineering News-Record.*)

How to detect the existence of permanent strain in metals is of considerable interest and often of decided importance to engineers. The term « permanent strain » necessarily implies a yielding of the metal of a structural member under the stress imposed upon it. So long as the metal behaves elastically under stress — that is, so long as the direct proportionality between stress and strain is not disturbed — no record remains to indicate either the magnitude or the distribution of a stress after this has been removed. If the stress is great enough, however, the material is permanently affected and the resulting change in the structure of the metal serves as a record of the previous treatment of the material.

One of the most important points in connection with the permanent straining of metals, at least in the initial stages — for example, during the « yielding » of mild steel — is the fact that this effect is usually quite local in character instead of being distributed uniformly throughout the strained member. It is this localization of the effect which constitutes, in large measure, the record of the change which has been brought about in the material.

Iron is the metal which shows this phenomenon best, and nearly all of the studies that have been made along this line have been on low-carbon and mild steels. This is of especial interest to engineers, since steel of this kind enters so widely into engineering work. The discussion which follows relates to this class of material.

Mechanism of straining. — The mode of deformation of any metal under stress is intimately related to and, in fact, is determined by the structure of the metal. The fact that from the structural point of view every metal is an aggregate of microscopic crystalline grains is so well established that further emphasis here on this point is unnecessary. These grains, except in very brittle metals or alloys, are capable of a change of form when stressed sufficiently, the change being brought about by minute *slipping of parallel layers* within the crystal, along definite crystallographic planes. Within any grain there are certain crystallographic planes of easy slip, this condition being dependent upon the crystal habit of the metal. Deformation or slip occurs most readily within any grain when the imposed stress acts along the planes of easy slip. The summation of the minute slippages occuring within a great number of the grains constitutes the visible elongation, or other deformation, which is measured on a test specimen — for example, a tension specimen.

The effect of stress on the structure of mild steel and the various methods for detecting strain-effects are most conveniently considered by reference to the curve summarizing the behavior of this material when stressed in tension. It should be borne in mind that the properties of any metal, as ordinarily determined, are those of an aggregate of grains, and not those of the individual grains. The properties of a single crystal (that is, a grain) of iron are distinctly different from those of the aggregate of which the grains form the units. In most respects the tensile properties of the single grains are lower than the corresponding properties of the aggregate. Particularly is this true for the elastic properties of the metal.

Elastic deformation. — So long as the proportional limit of the metal is not exceeded when the material is stressed, the structure of mild steel is not changed, at least not to an extent that the microstructure, after the load has been removed, is visibly affected.

Lester and Aborn (« The Behavior Under Stress of the Iron Crystals in Steel », *Army Ordnance*, vol. 6, p. 120, 1925), have observed irregularities in the behavior of some of the crystal grains in steel when the material was stressed within the elastic range (of the aggregate), which they have explained as indicating slight slip within the grains. The deformation is not great enough, however, to show in the microstructure of the material.

Strain effects accompanying yielding. — The strain effects which accompany the yielding of mild steel will be referred to here as « initial strain effects » in contradistinction to the effects which occur as the material is plastically deformed when the stress is increased above that corresponding to the yield point. The latter effect, which is progressively more and more severe and culminates in the fracturing of the specimen, is much more pronounced than the initial strain effect and may well be referred to by the term cold working.

A knowledge of the initial strain effects, the means for revealing them and their possible influence on other properties of the metal is much more valuable to engineers, as a rule, than a consideration of the more severe strain effects of plastic deformation or cold working, since the former often constitutes the first clear indication that the factor of safety for any given structure has been exceeded.

Surface strain markings have long been of interest to engineers and metallurgists. As early as 1836, Piobert observed peculiar markings on metal parts of guns after service. (*Mémorial de l'Artillerie,* vol. 5, p. 505, 1926. See also C. Frémont « La Cause de la formation du palier dans l'essai de traction des aciers doux », *Génie Civil,* vol. 82, p. 176, 1923.) About twenty years later similar observations were made by Lüders, and the phenomenon is usually referred to by his name, though often coupled with that of Hartmann, who, considerably later, studied the phenomenon in considerable detail. (See W. Lüders, « Ueber die Aeusserung der Elasticität an stahlartigen Eisenstäben und Stahlstäben », *Dingler's Pol. Jour.,* vol. 155, p. 18, 1860; and L. Hartmann, « Sur la distribution des déformations dans les métaux soumis à des efforts », *Comptes rendus,* vol. 118. p. 520.)

Figure 1 shows strain markings of this kind developed on the face of a steel column by subjecting it to compression. This column was one of a series built especially for test purposes and intended as representative of those used in the construction of the Delaware River bridge. (R. S. Johnston, « Compressive Strength of Column Web Plates and Wide-Web Columns », Bureau of Standards Technologic Paper 327, 1926). The fact that the strain was not uniformly distributed throughout the entire structural member when stressed in this manner is very evident. Those portions to which the yielding of the metal under

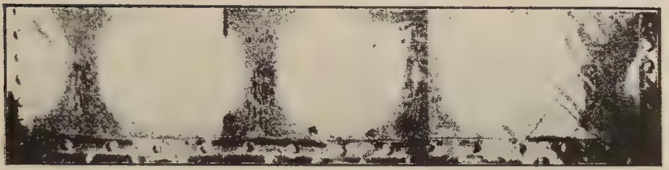


Fig. 1. — Strain markings developed in a steel column.

From Bureau of standards technologic paper 237, by R. S. Johnston. The surface which was covered with mill scale, was whitewashed with cement wash to secure good contrast when flaking of the oxide in the strain markings occurred. The specimen was a full-size tower web section of the Delaware River bridge, of silicon-manganese steel C, 0.33 °/o; Mn, 0.94 °/o; Si, 0.33 °/o.

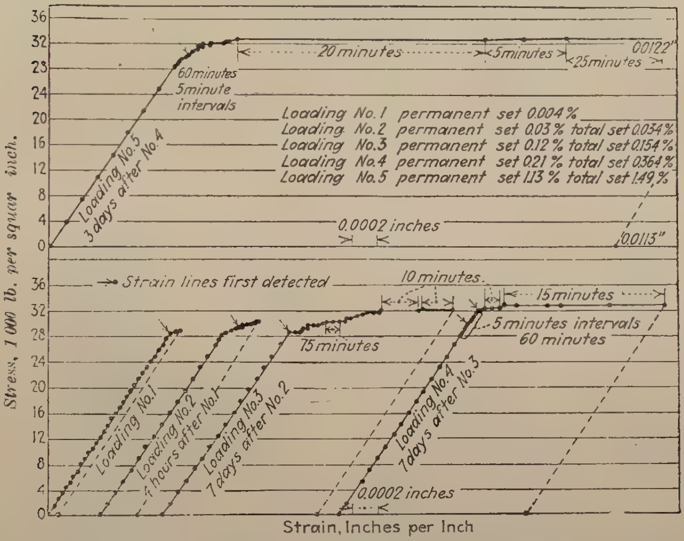


Fig. 2. — Stress-strain curves of mild steel stressed rcpeatedly in tension.

The formation of the strain markings coincides with the yielding of the material, and for this material agrees fairly well with the proportional limit. Composition of the steel was C, 0.17 °/o; Mn, 0.40 °/o.

stress was confined have been clearly revealed by the flaking of the surface oxide coating (mill scale) on the steel, together with the whitewash coating of portland cement in water which was applied over the oxide coating in order to increase the contrast between the flaked and the unflaked areas and thus delineate the strain markings more clearly for photographic purposes.

Observations made in this manner are most valuable in a qualitative way for demonstrating the variations in the distribution of the strain effect in a member which has yielded under load. In order to obtain reliable indications of the stress at which yielding occurred it is better to rely upon observations made on the polished surface of a test bar as it is subjected to a progressively increasing load. A determination of the yield point in tension of mild steel made in this way, as shown in figure 2, corresponded very closely to the proportional limit of the same material as determined by strain measurements made with a Ewing extensometer. The surface strain markings produced at one end of the specimen as it was repeatedly loaded are illustrated in figure 3, while figure 4 summarizes these markings for the entire specimen and shows how the strain effect or «yielding» progressed throughout the specimen from both ends as the loading was continued.

It will be seen from these illustrations that the first evidence of permanent strain or yielding in a specimen stressed in tension appears at the ends of the reduced section close to the fillet. This might be predicted, since, as can be shown by stress analysis by the optical method. (E. G. Coker, « Photo-elastic Methods of Testing », *Report* of British Asso. Advancement of Science, vol. 92, p. 313, 1924; also E. G. Coker, « Photo-elasticity for Engineers », *Gen. Elec. Rev.*, vol. 24, p. 82, 1921), the stress in this portion of a tension bar is higher than elsewhere throughout the reduced section. When the load is reapplied after it has been released in order that the specimen may be removed from the machine for study and for photographing, the strain effect continues where it left off at the previous loading. This continues with each successive loading until the entire reduced section has yielded to the stress.

It is fascinating to watch a polished tension specimen while it is maintained under load just sufficient to cause the formation of strain markings at the moment of yielding. The pronounced localization of the yielding; the very definite boundaries which separate the permanently strained portions from the part of the specimen which, as yet, shows no evidence of having yielded; the slow, often intermittent, advance of this boundary as the condition of permanent strain advances into the « unstrained » metal, all serve to emphasize the progressive nature of the process of the yielding or permanent straining.

When magnified slightly, the surface strain markings are found to have a somewhat rippled appearance, as is shown in figure 5. A previously polished surface takes on a matt finish within the strain markings, on account of the rippled surface which forms as yielding occurs. It will be noted that the ripples are roughly parallel to one another and approximately at right angles to the direction in which the stress was applied. As the specimen yields more and more, the strain markings widen and form branches; new strain markings also appear, usually just ahead of the pre-existing ones (that is, in the direction in which the strain effect is advancing). The size of the ripples on the surface of the strained areas is very much larger than the grain size of the material. Each elevation or « ripple » apparently is made up of a large number of grains.

Etching methods. — An etching method for revealing strain effects in mild steel has been described by Adolf Fry (« Kraftwirkungsfiguren in Flusseisen dargestellt durch ein neues Aetzverfahren, *Kruppsche Monatshefte*, 1921, vol. 2, p. 117; *Stahl und Eisen*, vol. 41. p. 1093, 1921), and is usually referred to by his name. This etching reagent consists essentially of copper chloride (or copper ammonium chloride) dissolved in an aqueous solution of hydrochloric acid. Those portions of a mild steel which have been permanently strained are attacked more readily than those parts which have not yielded under stress. This is well shown by figure 6, which depicts some results obtained by J. D. Jevons (« Strain Detection in Mild Steel by Special Etching », *Jour. Iron and Steel Inst.*, vol. 111, p. 191, 1925) with this reagent on a series of mild steel bars which

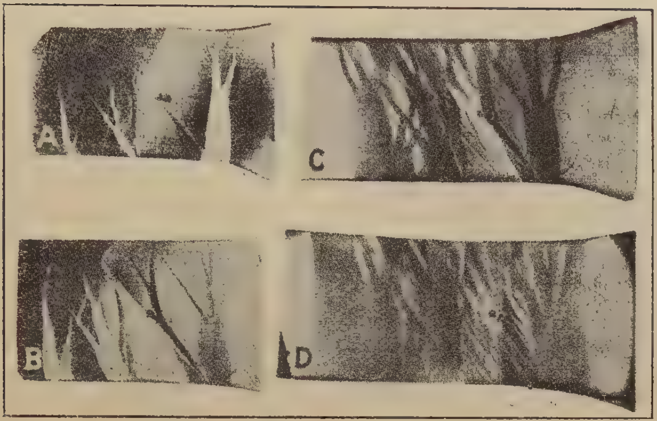


Fig. 3. — Strain markings on a tension specimen.

The part shown is one end of the reduced section of a specimen of fig. 2.

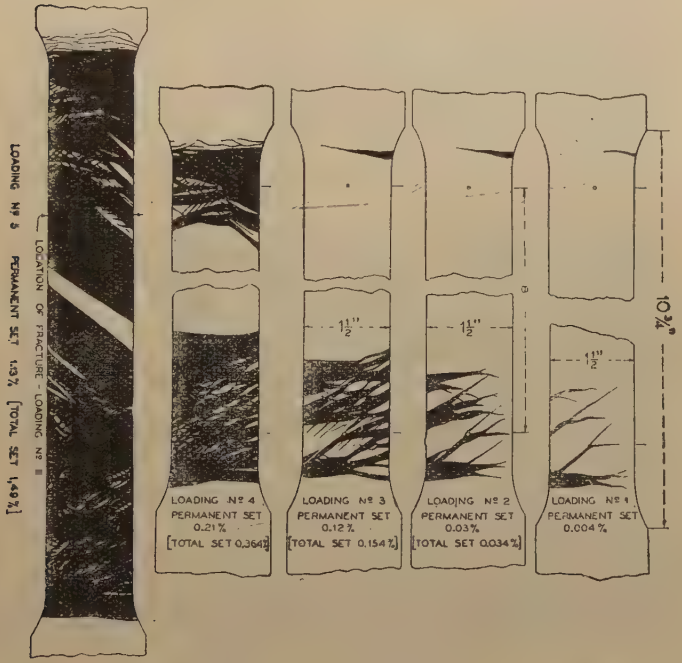


Fig. 4. — Progressive yielding.

Tension specimen loaded several times in succession; compare fig. 2.

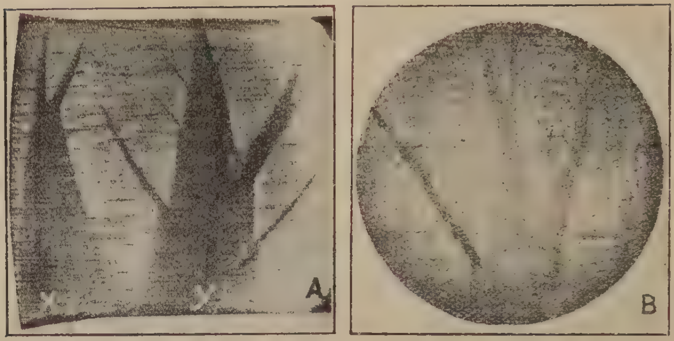


Fig. 5. — Surface appearence of the strain markings on soft steel.

In left-hand view the line *x* formed at 28 000 lb./in.^{2}; line *y* at 30 200 lb./in.^{2}; the load was held constant ten minutes and the wedges widened to the extent shown : the 45° branches formed at 32 000 lb./in.^{2}. Magnification : 2.

The right-hand view shows part of the specimen B of fig. 3. These markings were formed during loading No. 2, fig. 2. Magnification : 3 1/2.

Note the rippled surface within the strain markings, indicating the yielding of the metal.

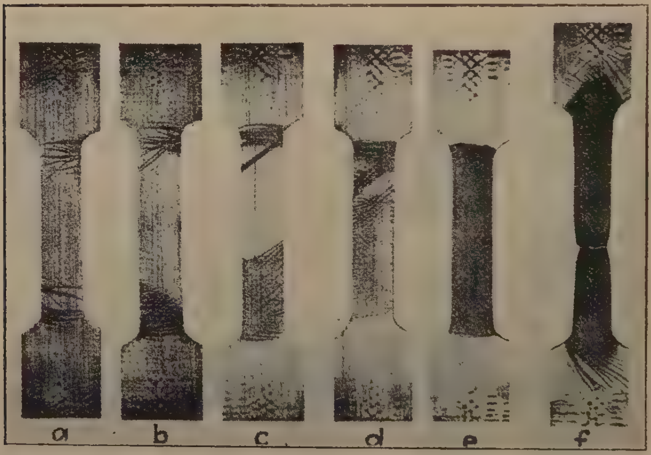


Fig. 6. — Steel specimens stressed in tension to different degrees and etched.

Acidified copper solution (Fry's reagent) used to show the strain distribution (J. D. Jevons, (*Journ. Iron and Steel Institute*, vol. III, 1925). The specimens were of 8-inch gage length ; they were strained and then machined smooth before etching.

a, « Yield point » ; *b*) 0.5 % elongation ; *c*) 1 % elongation ; *d*) 2 % elongation ; *e*) 4 % elongation ; *f*) broken at 29 % elongation.

had been permanently strained in tension by different amounts. These results, which were obtained by etching machined sections of the strained specimens and thus represent the condition of the interior of the specimens, are in very

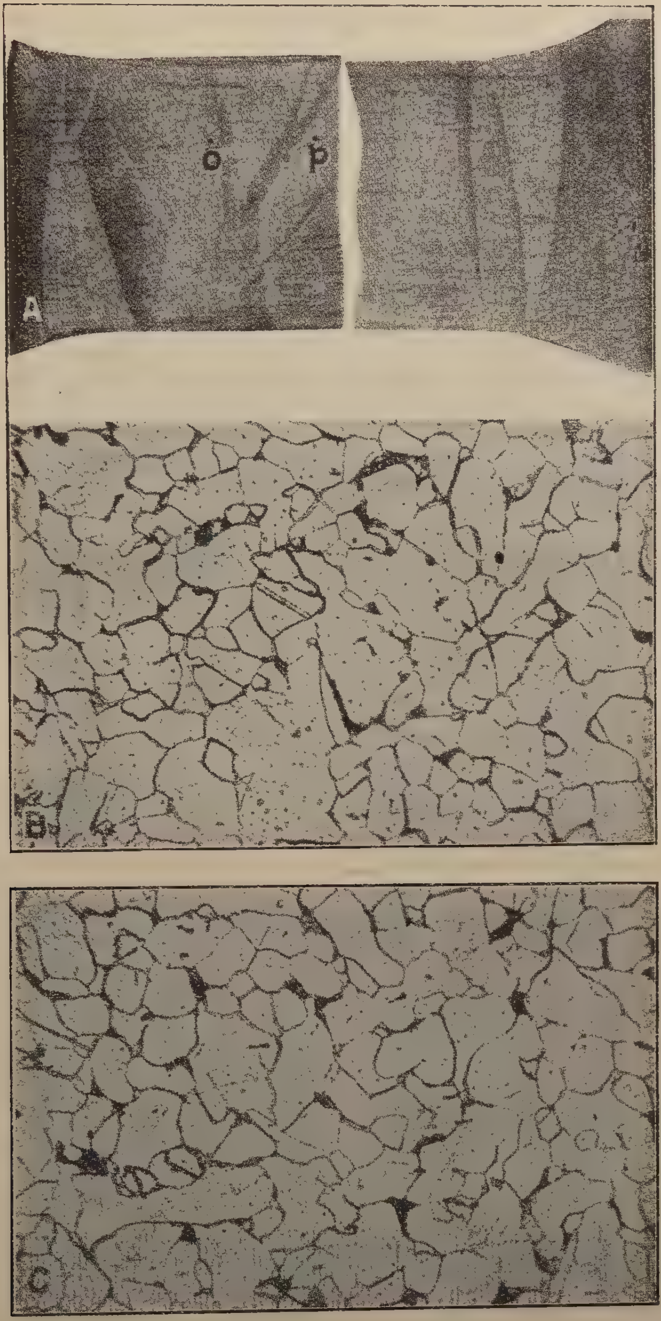


Fig. 7. — Microstructure of mild steel stressed until yielding occurred.

A. Appearance of an 18-inch specimen with strain markings developed at the two ends; full size.

B Structure corresponding to the point *O* — that is where the metal « yielded »; magnification : 250.

C. Structure corresponding to « unstrained » metal at the point *P*. There is no noticeable difference between the two.

close agreement with the surface strain markings shown in figures 3 and 4. The gradual and progressive yielding of the metal as the strain effect, which begins at the ends of the reduced section close to the fillet, advances along the length of the reduced section is clearly shown. Not until relatively late in the test does the process of yielding extend into the shoulders of the test specimen. Figure 6 also shows the strained condition near the ends of the specimens which resulted from the gripping of the specimen in the testing machine.

Microscopic examination. — It has frequently been reported that examination of the microstructure of mild steel which has been stressed just above the yield point shows nothing, as a rule, which can be considered as characteristic of material which has been strained in this manner. If the microscopic examination is made on sections cut from the strained bar, polished and etched in the ordinary manner, no significant difference will be seen in those portions corresponding to the strain markings as compared with the unstrained portions. (See fig. 7.)

However, if the surface to be examined microscopically is prepared before the specimen is strained, the nature of the effect on the microstructure of the permanent straining can be shown, as is illustrated by figure 8. It will be seen from this illustration that many of the grains within the strain markings show distinct evidence of deformation by slipping, the slip bands (that is, the traces of the slip planes on the surface of the specimen) in one grain being approximately parallel to those in the other grains and at right angles to the direction of stress application. Not all of the grains show evidence of deformation by slipping; evidently this effect is confined to those grains which are favorably oriented crystallographically, with respect to the direction of the applied stress. Presumably slipping occurs first within those grains in which the stress acts along planes of easy slip.

It is evident, therefore, that the initial strain effect produced in mild steel by stressing it until yielding occurs is a case of deformation by crystal slip. Unlike the more pronounced plastic deformation which occurs as the material is stressed well above the yield point, the effect seems not to be general for all of the grains. Only part of the grains respond during the yielding of the metal, and in none of the grains is the effect pronounced enough so that the form of the grain is noticeably changed.

X-ray examination. — The X-ray examination of strained metals has proved to be an exceedingly useful means for the study of strain in metals, especially for showing how this condition is related to the crystal structure — for example, which of the crystallographic planes are concerned mainly in the phenomenon of deformation by slipping and how the condition of « preferred orientation » in severely deformed metals is brought about. In the case of steel which had been stressed only sufficiently to produce the initial strain effects, however, an X-ray examination (fig. 9), failed to give any clear indications as to the nature of the change which had been brought about. Evidently this method is not a suitable one for demonstrating whether or not a steel has been stressed to the yield point; a more severe strain effect, as is shown by the ends of the specimen where it was gripped for carrying out the tension test (fig. 9*c*) is necessary in order that clear indications can be obtained by the X-ray method.

Strain beyond the yield point; cold working. — When mild steel is stressed beyond the yield point, the reduced section of the specimen becomes quite uniformly strained along its entire length, as is shown at *e* in figure 6. The preexisting initial strain markings on the surface formed during the yielding of the metal (figs. 3 and 4) are almost entirely obliterated by the relatively high flow of the metal which takes place under the higher stress. Etching the surface of such a severely strained steel with Fry's reagent serves mainly to show that such parts have been uniformly strained.

Microscopic examination of such metal shows a marked change in the form of the grain. As the deformation (in tension) is progressively increased, they become longer and, if the deformation is very severe, as in wire-drawing, the grains assume the form of fibers. This change in the grain form is illustrated in figure 10. X-ray examination of such severely strained metal, unlike that of the same metal stressed only to incipient yield, shows a marked change in the X-ray pattern. This method not only has proved very useful in clarifying our knowledge as to what happens during the very severe deformation of metals but has many more practical applications, such as in the detection of a condition of internal strain in metals resulting from manufacturing operations and the treatment necessary for their removal.

If severely strained steel is annealed, marked changes in the structure occur which, to the metallurgist, tell a good deal concerning the previous condition of the material. The changes are mainly those of recrystallization and grain growth, and the subject, which is quite involved, need be considered here only very briefly. All tests of this kind necessitate cutting specimens from the strained metal and subsequently annealing them. Steel which has been strained only sufficiently to produce yielding has, in tests carried out by the writer, shown no tendency toward recrystallization and grain growth upon annealing. On the other hand, steel which has been strained by stressing it considerably beyond the yield point shows an entire change in its microstructure after being annealed. The amount of the deformation or permanent strain produced, the annealing period and temperature, are all involved in the change produced in the microstructure. The method is particularly adaptable in those cases in which steel has been differentially strained. This is illustrated by figure 11, which shows the grain structure of a tapered specimen of mild steel which, after being permanently strained in tension by stressing it until fracture occurred, was annealed. The marked variation in the grain size according to the degree of strain produced is very evident, as the illustration shows.

Strain lines produced by shock. — A stress of the nature of impact or shock often produces a strain effect in iron and mild steel which is distinctly different from that produced by static stress. This effect, which is shown in figure 12, is a microscopic feature and consists in the production of narrow bands extending across a grain. These bands, usually referred to as Neumann bands, are generally believed by metallurgists to partake of the nature of twinned crystals. The iron atoms within these bands are believed to have been changed sufficiently in their position by the shock so that, with respect to the unaltered portion of the grain, they bear the same relation that twin crystals do to each other. The sparkling crystalline fracture which steel broken by impact often exhibits undoubtedly is another manifestation of this behavior of iron under shock. The production of Neumann bands under impact seems to be peculiar to iron and mild steel — that is, steel containing a considerable amount of ferrite (iron).

Miscellaneous strain effects. — It is well known that metals which have been permanently strained are harder than the same metal before straining. The increase in hardness produced depends upon the degree of deformation. The

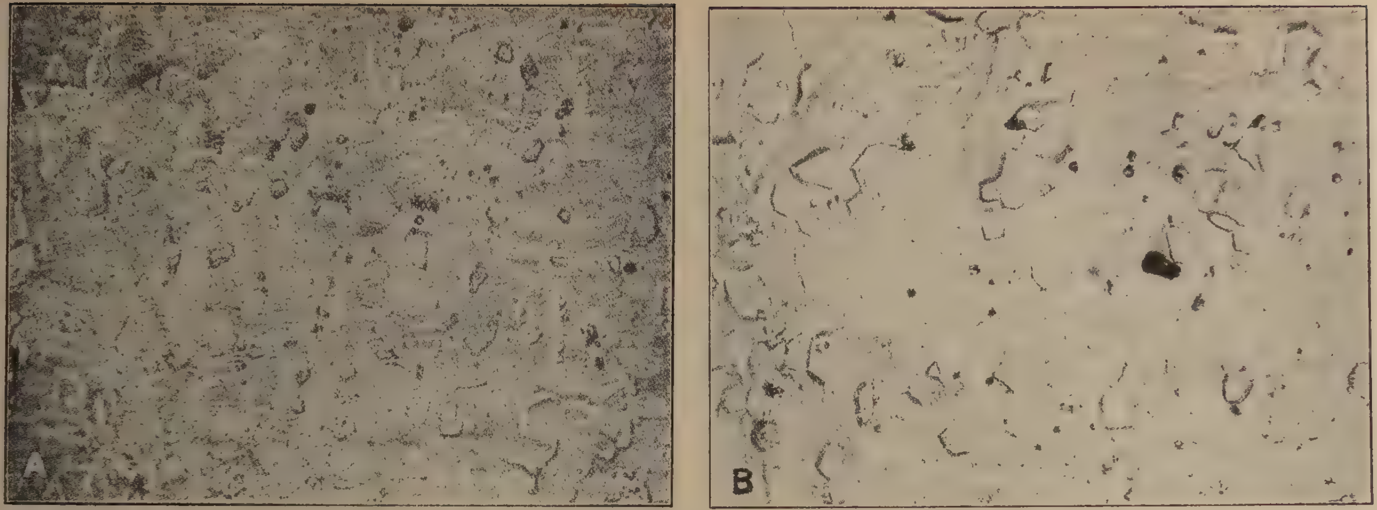


Fig. 8. — Effect of stressing mild steel until « yielding » occurs.

Structure as shown by the surface examination of a specimen polished and etched before straining; magnification, 250.
A. Structure within the strain markings. *B*. Structure of immediately adjacent metal which had not yielded under stress.

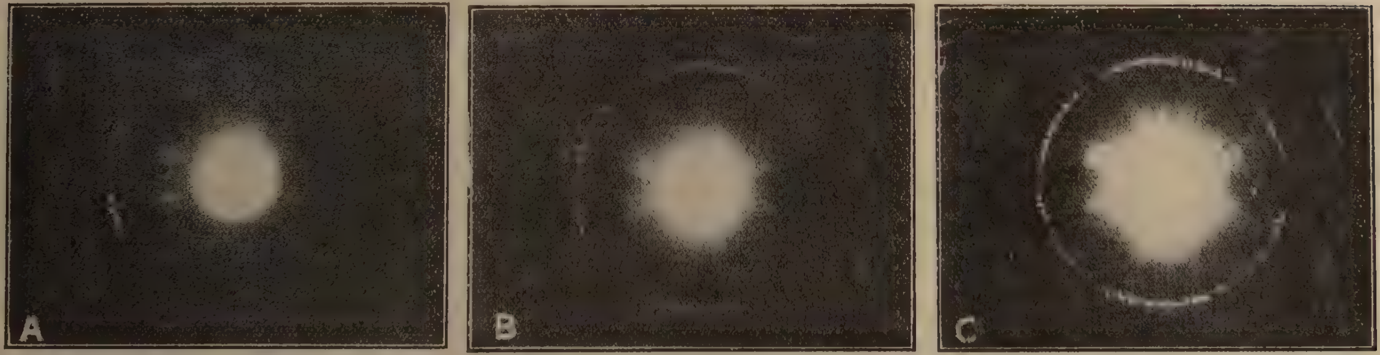


Fig. 9. — X-ray patterns of stressed steel.

The X-ray beam was passed through (Laue method) a specimen of thin steel strip which had been stressed in tension.
A. X-ray pattern of unstrained steel immediately adjacent to the strain markings.
B. Similar pattern of steel which was permanently strained — that is which « yielded » under the applied load.
C. Similar pattern of metal at the end of the specimen where it was gripped.
Note that *A* and *B* are essentially alike. — The feature shown in *C* (asterism) is indicative of severe strain.

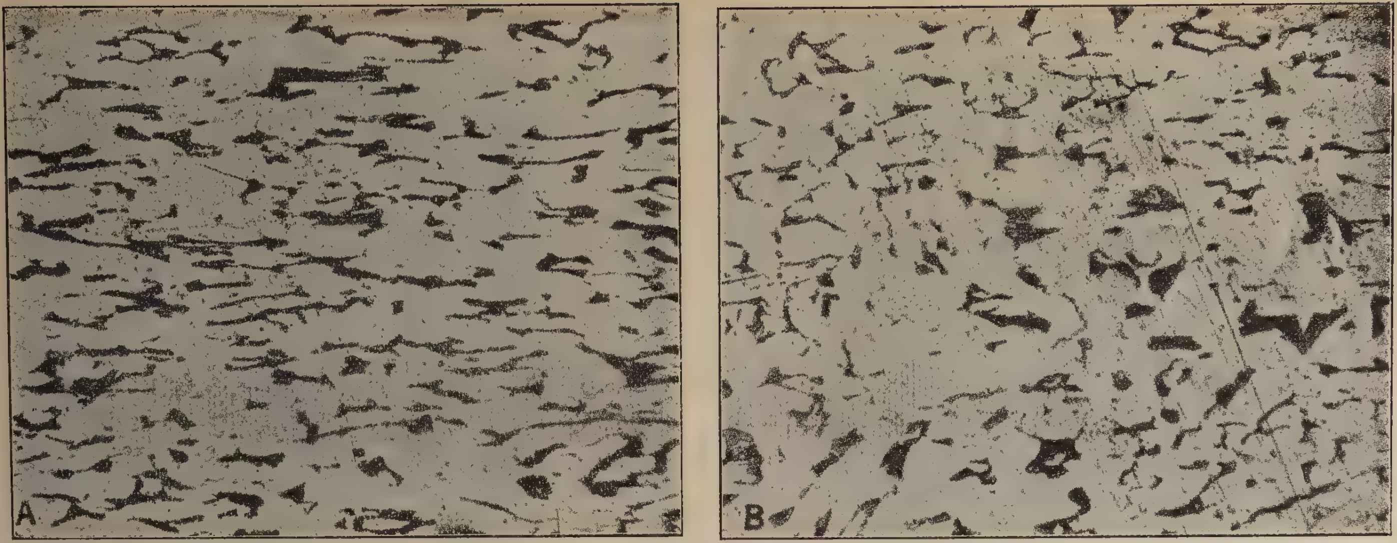


Fig. 10. — Effect of cold working on grain structure.

An 18-inch tension specimen similar to that of fig. 7 was stressed until fracture occurred. Magnification, 250. The micrographs show : A the condition at the fracture and B about 2 inches from the fracture. Fig. 7 shows the original grain structure.

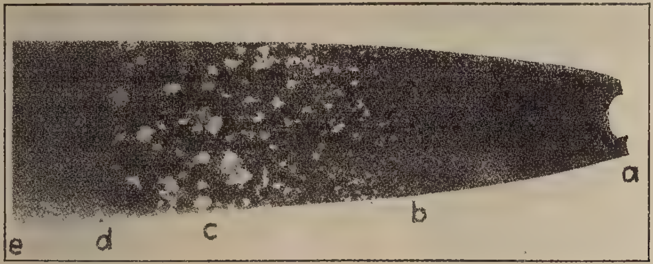


Fig. 11. — Effect of annealing.

Grain structure of a severely strained steel specimen in which the degree of strain varied along its length. A tapered bar which had been broken in tension was annealed six hours, 1 265° F (685° C) and then air cooled. Maximum stress on the bar at *a*, 49 300 lb./in.^{2}; at *b*, 40 300 lb./in.^{2}; at *c*, 35 700 lb./in.^{2}; at *d*, 34 000 lb./in^2 : at *e*, 32 500 lb./in.^{2}. Yield point of the steel as shown by strain markings, 26 800 lb./in.^{2}.

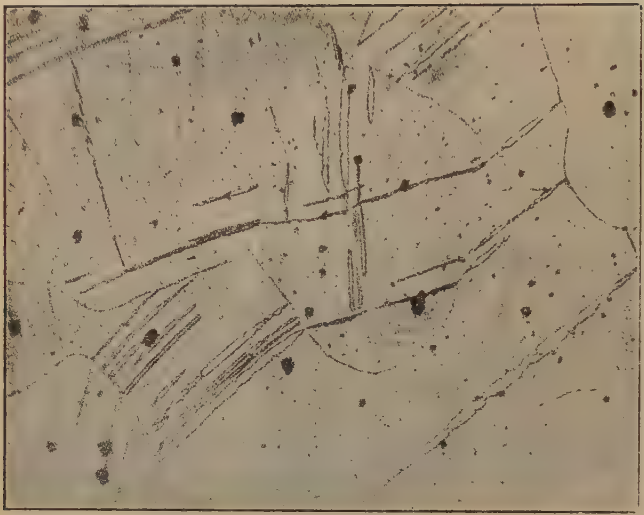


Fig. 12. — Strain lines developed by shock.

The micrograph, magnification 250, shows the structure near the fracture of a specimen of open-hearth iron broken by the single-blow notched-bar impact test. The narrow bands (Neumann lines) which extend across the grains are characteristic of the effect of very suddenly applied stress on iron and mild steel.

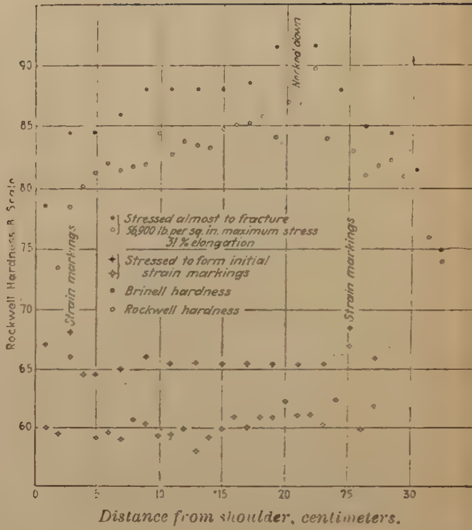


Fig. 13. — Hardness survey of a stressed steel ba

Lower points for tension bar stressed to incipient yie
as shown by the formation of strain markings. Upp
points for bar stressed until « necking down » occurred ju
before fracture.

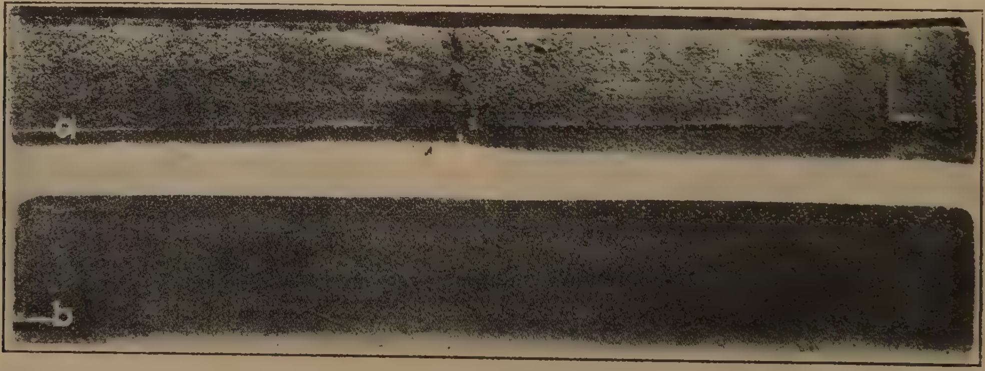


Fig. 14. — Effect of localized strain upon the corrodibility of steel.

a) Strip cut from the end of a boiler tube which was perforated by corrosion within a few months after installation.

b) Similar strip from the same end of the tube, machined flat and etched with Fry's reagent. Note the diagonal lines indicating strain. The anodic behavior of this strained portion with respect to the remainder will account for the severe localized corrosive attack.

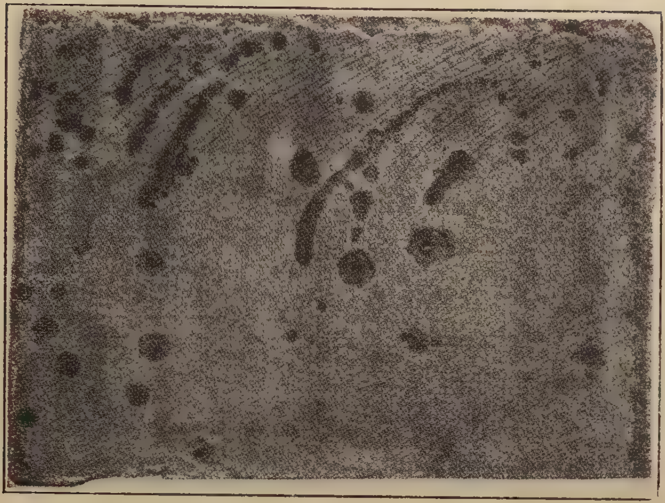


Fig. 15. — Effect of strain on the corrosion of steel.

Note how most of the corrosion pits coincide with the strain lines which were set up by the cold shearing of the 1-inch plate. The greater part of the surface of the plate, away from the edges, showed only a general corrosive attack. The strain lines were revealed by etching with Fry's reagent.

effect upon the hardness of mild steel produced by stressing it in tension until yielding began is shown in figure 13. The results summarized clearly show that localized hardening of the metal accompanies the yielding of the metal at the ends of the reduced section close to the fillets. Elsewhere the hardness has not been appreciably affected. If the load were maintained long enough so that the metal of the entire reduced section responded by yielding, it would be found that all of this portion of the specimen had been hardened to the same degree as is indicated by the two peaks in the hardness survey of the specimen, which was stressed until yielding had just started.

The hardening effect of stressing the same material until fracture almost occurred is also shown in figure 13. It will be seen that the hardening accompanying the yielding of the material is slight as compared with that produced by stretching the material nearly to the point of fracture; the maximum increase in hardness produced in the latter case was about ten times that in the former.

In general, the corrosion-resistance of strained metal is less than that of the unstrained metal. This is best shown if the strained metal is in metallic contact with the unstrained specimen; when wet with an electrolyte — for example, salt solution (sodium chloride) — the strained metal behaves as an anode with respect to the unstrained metal and therefore is corroded. The effect is still more striking and in actual service very much more serious in the outcome, if the same piece of metal has been treated so that it has been permanently strained locally — that is, the strain was confined to a limited portion of the specimen. Figure 14 illustrates how the corrosion of a boiler pipe was localized in that part which had been deformed by hammering, the attack being so severe that perforation of the wall of the tube resulted within a few months. In figure 15, the course of the corrosive attack of a steel plate by sea water will be seen to have been directed to a large extent by the strain lines produced during the shearing of the plate.

Summary.

Certain conclusions as to the effect of strain upon mild steel may be drawn from the examples cited in the foregoing. The principal ones are :

1. The effects of strain upon metals, as recorded in the structure, have been considered for a mild steel. Similar effects are noticed, however, as a rule, in any steel containing a considerable amount of ferrite.

2. Elastic strain produces no change in the microstructure of a steel.

3. The changes corresponding to « yielding » under stress, which are of particular interest to engineers, are very evident and may be demonstrated :

a) by the strain markings on polished surfaces or the flaking of oxide or other coatings;

b) by macroscopic etching with acidified copper solutions;

c) by microscopic examination of polished surfaces subsequently strained, and

d) by hardness surveys. X-ray examination is of very doubtful value for determining whether or not a steel has yielded under load. The strain effect of « yielding » is decidedly localized and progresses gradually throughout the reduced section as the load is maintained.

4. Stress beyond the yield point produces a progressively greater change in the appearance and the structure of steel. The surface, which becomes completely freed from any scale initially present, assumes a matt appearance, entirely obliterating the strain markings produced during the first yielding of the metal. The grain structure is greatly changed and a fiber-like structure is approached with high degrees of straining.

5. The effects of impact and shock stress on steel are recorded in the microstructure in characteristic and definite manner.

6. The corrodibility may be greatly increased as a result of permanent strain. This effect is likely to be most serious, from a practical standpoint, when the piece has been only locally strained, as in the case of the boiler tubes given herein.

[621 .135. (01) (.42) & 621 .135.2 (.42)]

Flanges of locomotive tyres.

Figs. 1 to 7, pp. 190 to 193.

(*The Railway Gazette.*)

In Col. Mount's report on the accident that occurred at Bridgnorth on 13 January 1928, when the locomotive and following coaches became derailed, he calls attention to the fact that the Great Western Railway Company are making the flanges of their locomotive wheels slightly deeper, in the following terms :

« In my opinion this is not an example of spreading to such an extent that at the point of derailment the tread of one wheel dropped inside the rail, thus bursting the road. It is of interest to note that the profile of the gauge edge and rail table was normal, and there was, therefore, nothing in that aspect to contribute to the result. The tyres were, also practically unworn and the flanges (of the leading and trailing coupled wheels) to the standard or « thick » profile (as compared with the « thin » profile of the driving wheels) 1 1/8 inches in depth. The case, therefore, shows, so far as it is possible to draw conclusions upon surmise, that in respect of these two factors — the rail and the flange — the only thing which might perhaps have prevented the derailment, under the pertaining adverse track conditions, would have been the existence of deeper flanges providing a better angle of contact with the rail.

« At any rate, the risk would have been lessened, and I think it of advantage to draw general attention to the fact that this company has adopted since August last (as the result of certain derailments on the system and elsewhere) what appears to be an eminently desirable policiy of deepening all engine flanges by 3/16 inch. This is being obtained by either turning up the treads of existing tyres or by having new tyres rolled with the deeper flanges, the present standard dimensions in respect of the point being retained. I understand that no difficulty is being experienced in regard to contact with permanent-way fittings, maximum depth being limited to 1 1/2 inches instead of 1 5/8 inches, the existing permissible limit of wear being thus reduced from 1/2 inch (1 5/8 inches minus 1 1/8 inches) to 3/16 inch (1 1/2 inches minus 1 5/16 inches), which more nearly represents the maximum tolerance permitted in practice, treads not being allowed to become hollow by more than this amount. »

We have been in communication with the Great Western Railway Company and find that Mr. C. B. Collett, the Chief Mechanical Engineer, has been giving careful consideration to the question of locomotives tyres.

The present standard of locomotive tyres, as published by the British Engineering Standards Association in 1927 (No. 276), is practically identical with that which was previously adopted on the Great Western Railway. We gather that Mr. Collett has kept the civil and

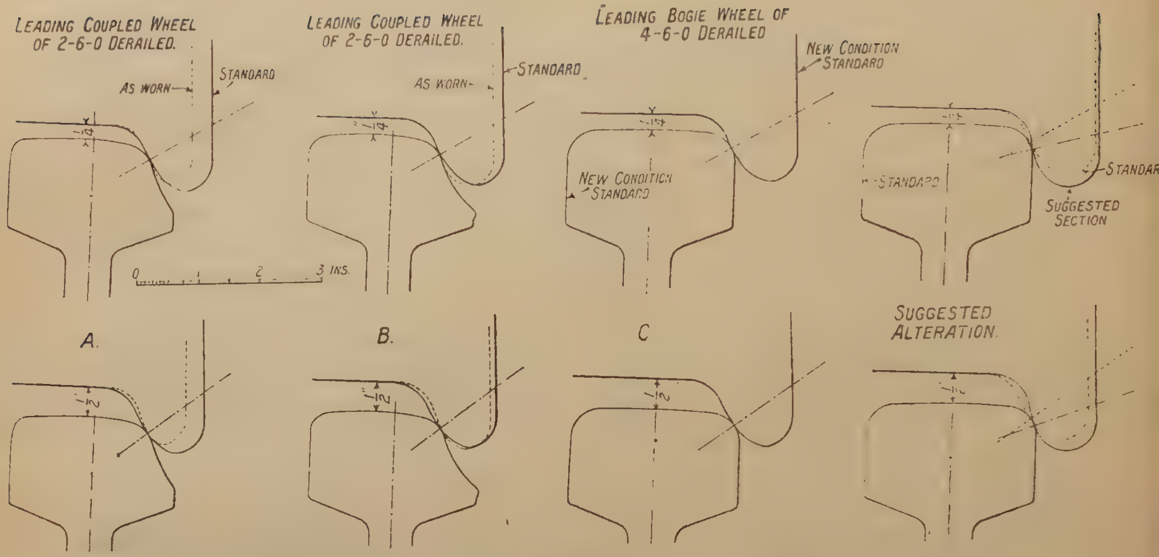


Fig. 1. — Tyre and rail sections at recent derailments and suggested alteration to standard flange.

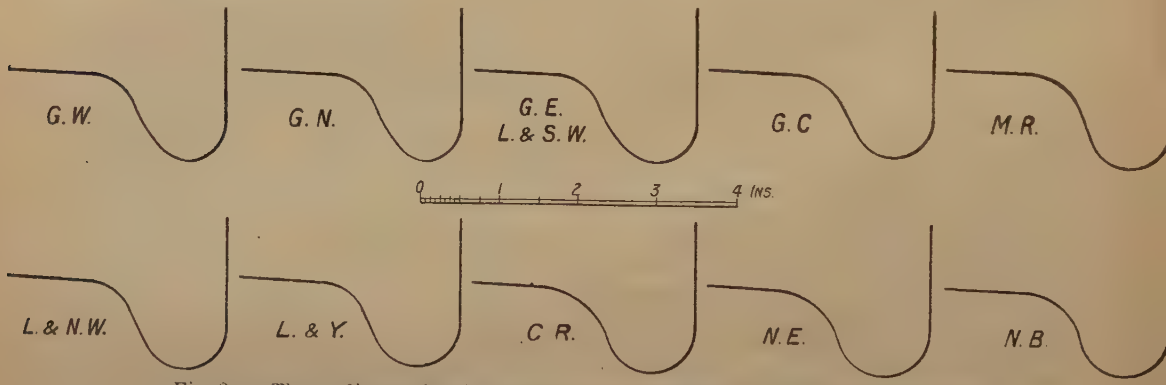


Fig. 2. — The profiles employed by constituent Companies in the four grouped Railways.

mechanical engineers of the other railways in touch with the alterations he has made.

Although the recent derailments on curves have no doubt been primarily due to other circumstances, the shape of the standard tyre is perhaps not that best suited for resisting heavy side pressure at high speed, both on account of its shallowness and of the angle of contact between the flange and the rail, particularly when the rail has become slightly worn or when the track is laid on a spongy foundation. In the derailments which have taken place the tyres have been practically full new section, and it seems unreasonable that engines turned out with tyres in new condition should

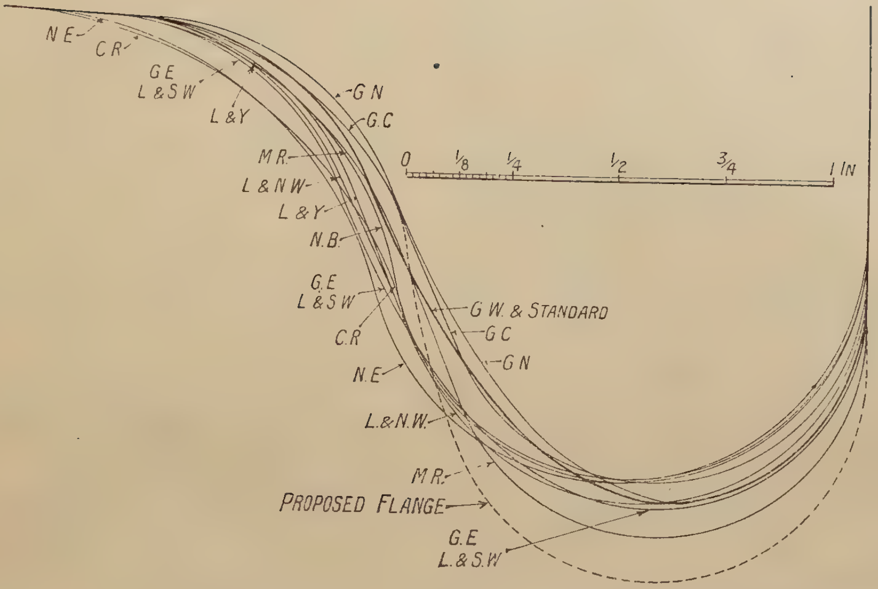
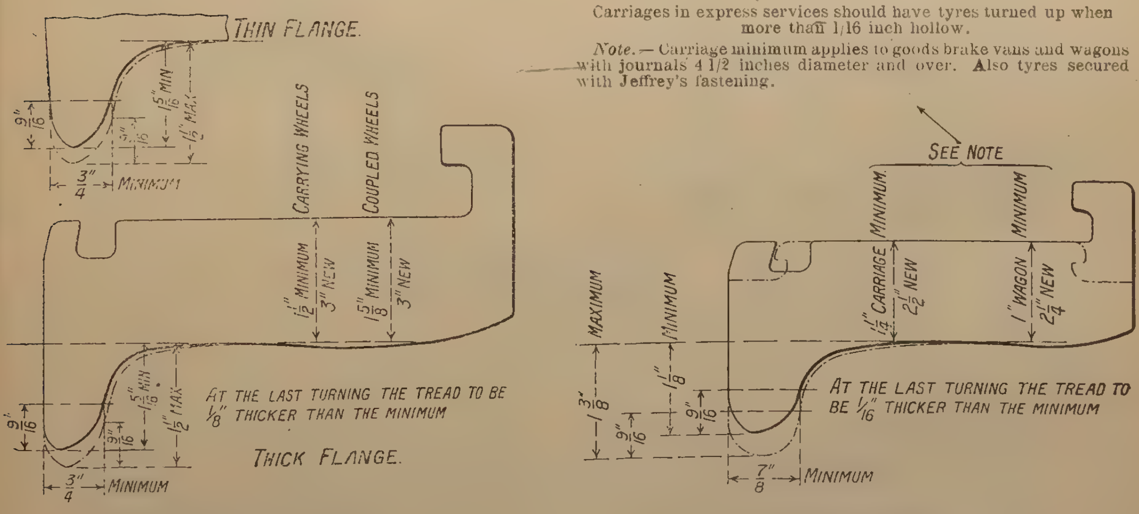


Fig. 3. — Tyre profiles superimposed, including Great Western Railway suggested flange.



Carriages in express services should have tyres turned up when more than 1/16 inch hollow.

Note. — Carriage minimum applies to goods brake vans and wagons with journals 4 1/2 inches diameter and over. **Also** tyres secured with Jeffrey's fastening.

Locomotives and tenders.

Carriages and wagons.

Fig. 4. — Diagrams showing the minimum thickness of flange and tread and maximum and minimum depth of flange of tyres.

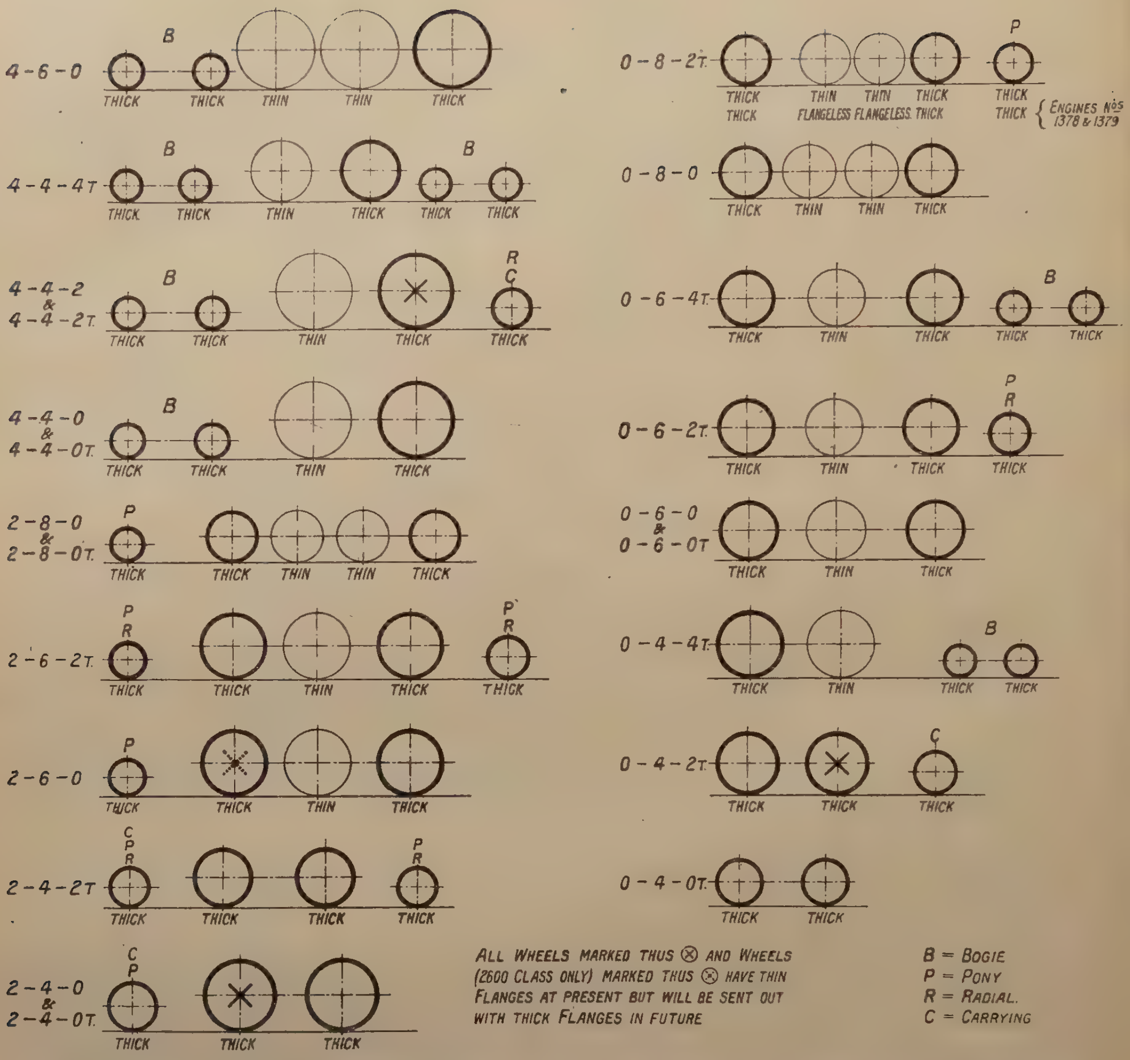
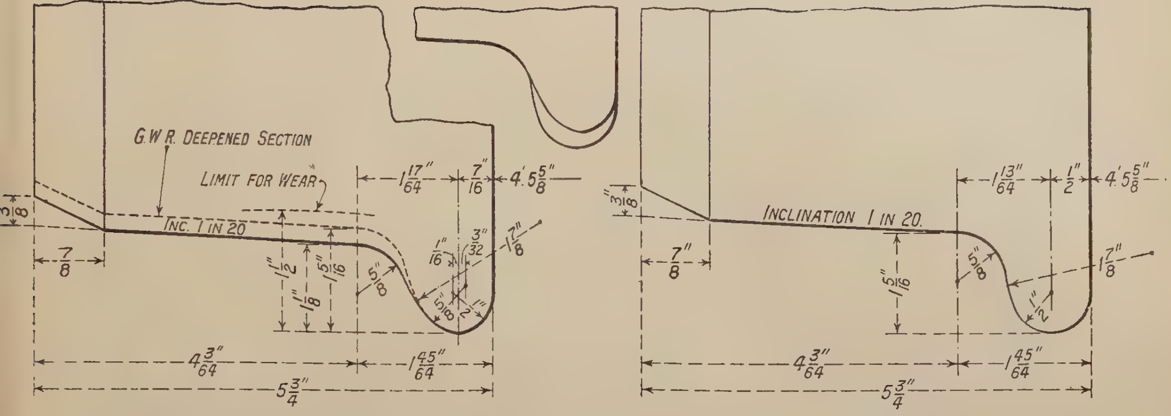


Fig. 5. — Diagrams showing the position of thick and thin flange tyres on Great Western Railway locomotives.

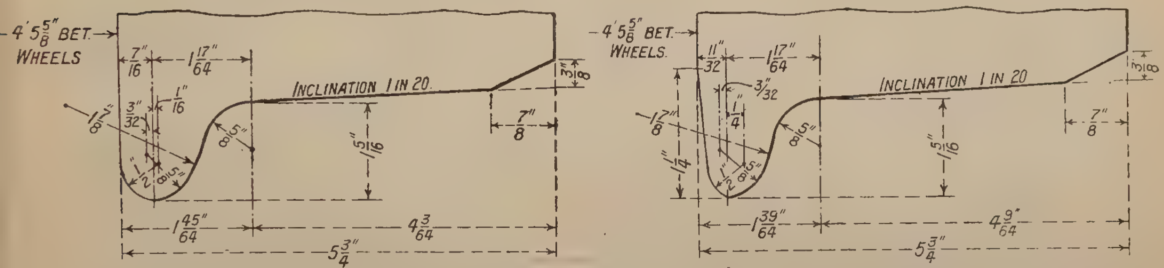
be less safe than those that have become worn and are deeper. It is true that the present standard section is perfectly satisfactory under normal conditions, but with the present high speeds and heavier engines, especially when a track is suffering from extremely bad weather conditions, it might be made of a section more capable of resisting derailment and which would also tend to side-cut the rails less by deeper engagement and at such an angle as better to resist side pressure. There have, unfortunately, been numerous derailments both in this



Standard thick flange. B. E. S. A. Section A.

Proposed thick flange.

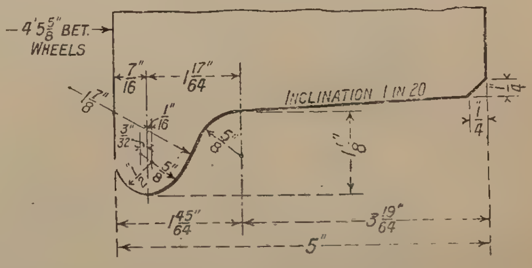
Fig. 6. — Comparison of standard and proposed thick flange tyre profiles.



Thick Flange, Locomotive Tyre.

Thin Flange, Locomotive Tyre.

Fig. 7. — Great Western Railway standard tyre profiles.



Carriage and Wagon Tyre.

country and abroad where apparently the locomotives have simply run off the track and usually at curves.

In our illustrations we show in figure 1 (with corresponding diagrams showing suggested alteration to standard flange) the actual condition of rails and tyres in three recent derailments, in two of which both rails and tyres had worn considerably, but in one both were practically to standard. In the first two there was no evidence that the track had given way, but in the last, where the formation was spongy in a boggy area, it is generally thought to have done so.

Figure 2 illustrates the various sections of tyres used on different railways before the new standard was adopted, and figure 6 shows how the Great Western Railway new flange compares with the standard at its commencing depth, which formerly was 1 1/8 inches after wear was allowed to reach 1 5/8 inches, and which now commences at 1 5/16 inches and is not allowed to wear deeper than 1 1/2 inches, so that, of course, the effect on rail fittings is nil. The flanges are only deeper to commence with and are really shallower to finish with, the limits between the minimum and maximum being tightened up. Figure 4 shows the minimum thickness of flange and tread and maximum and minimum depth of flange of tyres, and figure 7 shows new Great Western Railway standard tyre profiles for locomotives (thick and thin flanges) and carriages and wagons.

We find that the maximum depth in America is the same as that adopted by Mr. Collett, *viz.*, 1 1/2 inches, It is interesting to note that limits for the allowable wear of tyres have been introduced on the Great Western Railway, but we cannot trace that that or any other railway has done the same with regard to rails.

In view of the ease and facility with which this additional safeguard can be introduced, it would seem a very desirable practice to adopt, as this extra 3/16 inch, although a small dimension, makes a great deal of difference in the appearance of the tyre, and, as it amounts to 16.6 % increase in the depth of the flange, it must considerably add to the safety factor when through unexpected or unavoidable causes the track becomes a little irregular.

It will be noted that the shape of the standard flange having a 5/8 inch radius at the inside of the point has been adhered to in the deepened section in use on the Great Western Railway, and that in the suggested section a fuller point with 1/2 inch radius is proposed, more like that used on the Midland, North British, North Eastern, Great Central, London, Brighton & South Coast, Caledonian, and several other railways where the radius at this point was less than 5/8 inch, and in some cases as small as 3/8 inch.

Mr. Collett felt that it was undesirable to adopt a fuller point until some agreement was come to with the other railways, while the deepening that he is now working to is not a departure from the standard shape of flange, and can be carried out within the present limits of wear.

[621 .135.2]

The heating of locomotive axle bearings.

By H. B. BUCKLE, B. Sc. Eng.,
MOTIVE POWER SECTION, LONDON MIDLAND & SCOTTISH RAILWAY, DERBY.

Figs. 1 to 7, pp. 197 and 198.

(*The Railway Engineer.*)

Of all running repairs to which a locomotive is subject, it is probable that for none is it so long out of service as for heated axle bearings. To this must be added the fact that these heated bearings may and do cause delays to important trains. It will thus be seen that some consideration of this subject may not be out of place, for, while improvements in design and lubricants have had their share in minimising the trouble to a reduced extent, the trouble remains.

Fundamental considerations

The function of an axle-box bearing is to transmit the load that is brought to bear on it to the revolving motion of the axle journal that rotates in it. This operation is performed satisfactorily only when no undue wear of the surfaces occurs between which the movement takes place.

With the exception of crank journals, which will be discussed towards the close of these notes, all locomotive axle bearings share in carrying the weight of the locomotive itself, while some of them fulfil the further duty of transmitting the power exerted by the locomotive to the axle, and thence to the wheels. We have thus « gravitational » load bearings and « gravitational-power » load bearings, two groups which in practice are generally considered to need different treatment in design. The bogie and tender bearings clearly come under the first category, while the driving axle bearings with the bearings of all coupled wheels are in the latter class. The heating referred to in our title, which takes place in these bearings, is that which occurs when there is a tendency for seizing to take place between the surfaces of relative movement. It is the designer's duty to provide against this heating, and in due course we shall deal with this part of the subject. Under normal running conditions it is certain that these two surfaces do not come into contact, being separated from each other by the oil film of the lubricant; while to prevent heating before the oil film is created, some form of anti-friction metal is generally introduced into the journal bearing.

The bearings concerned with dead-weight only do not usually give much trouble so far as heating is concerned, but with the « gravitational-power », or, as we may call them, the « coupled » axle bearings, especially in the case of driving axles of engines fitted with Stephenson's link motion as applied to inside cylinders, there is only a limited space left for the bearings, and they are usually loaded up to what must be considered as a high load, a load in the case of certain engines which, with untoward circumstances intervening, may result in seizing.

The question of the actual loads on the

bearings may now be considered, though it is not thought advisable to enter in detail into the calculations which are necessary to obtain them. So far as the purely gravitational bearings are concerned the matter is simple, it being only necessary to take the load on any wheel (less the « deadweight », *i. e.*, wheels axle, axle-boxes, and springs), as given by the weighing machine, and divide this by the projected area of the axle-bearing surface. A fair figure for a tender bearing would work out at 350 lb. per square inch, though on some railways a much higher figure is reached. For the coupler axle bearings the matter is more complicated. The maximum cylinder load must be taken of the engine concerned, and from this must be plotted power diagrams, showing the push or pull on the piston. The effect of these on *each* axle-box can then be plotted on the same diagram. By taking moments the resultant thrust — a horizontal one — can then be found for each axle-box. If the effect of the side rods in adding to this thrust be taken into consideration, it will be found that for an engine speed of 20 to 30 miles per hour and with an engine with a right-hand crank leading, the thrust on the right-hand axle-box will be some 20 % greater than that on the left-hand box. The result of these calculations shows why in the case of engines with the right-hand crank leading it is generally the right-hand axle box which runs hot, and in engines with the left-hand crank leading, *vice versa,* it is the left-hand boxes which give most trouble. All who have experience with the heating of axle-boxes will know that the percentage of heated bearings with, say, an engine having a right-hand leading crank is much greater for the right-hand than for the left-hand driving axle-boxes.

To obtain the total resultant load on the bearings the piston thrust must be taken as explained in the previous calculation. This must be taken in conjunction with the gravitational load and the resultant obtained from the triangle of forces. From an actual case calculated from the indicator diagram of a four-wheel coupled passenger engine, and allowing for the effect of the coupling rod loads, the approximate figures work out at 650 lb. per square inch for the power load, and at 300 lb. per square inch for the gravitional load. The resultant force will produce a load of 716 lb. per square inch, but as the resultant line of this thrust is at only 24 3/4° with the horizontal, and as the bearing area in a locomotive axle bearing does not extend down to the horizontal centre line, it may be presumed that the actual load will be some figure even in excess of that given. In addition to this, it must be noted that the peripheral speeds of these journals frequently reach over 500 feet per minute.

There can be little doubt that were stationary engines subjected to the bearing loads of locomotives, seizing of the bearings would occur. The position must in some measure be helped by the road over which the locomotive works, the slight jarring over rail-joints possibly assisting in some way the desired distribution of the lubricant. We shall deal later with the question of method and point of application of the lubricant to the bearing, but it may be mentioned here that whatever the pressures between brass and journal, and undoubtedly, in many cases, they are very high, yet in the great majority of cases, with feeds to the crown of the brasses only, these axle bearings run without giving the slightest trouble. This statement is made with one reservation, for though as stated, the only feeds are the « top » feeds referred to, yet unless the boxes have their underkeeps padded, though the only oil these pads get is that which comes over from the « top » feeds, without them the trouble with heated bearings is much more pronounced. In the now historic experiments on friction in axle bearings, carried out under

Mr. Beauchamp Towers' supervision by a research committee of the Institution of Mechanical Engineers, and reported in the Society's Proceedings in 1885, it was shown that even with oil bath lubrication with pure rape oil, at a rubbing speed of 150 feet per minute, seizing of the bearing would occur at a pressure of 600 lb. per square inch. We can, therefore, only account for the locomotive axle bearings running with comparative immunity from trouble by some such explanation as that already given.

The design of the bearing.

In figure 1 will be found an example of an axle bearing for the driving wheels of an express passenger engine. It is of manganese bronze with gun-metal strips and white metal on the bearing surface, and has the advantage over a steel box fitted with a « brass », being wholly of similar metal of dissipating any heating thay may be produced more effectively. Particular attention should be given to the size of the white metal pads. These should be as large as possible, consistent with the strength of the brass, so as to reduce friction when the engine is starting, and the oil has not begun to function, to the minimum. It is well-known that lead is the best antifriction metal, provided that it will stand the load imposed on it, but, of course, in such bearings as we are dealing with pure lead is out of the question, as it will not stand the stress, but mixtures of metals which have been found to give satisfactory results are as follows :

A.R.L.E. No. 1 MIXTURE.		A.R L.E. No 3 MIXTURE.	
Coupled axle boxes.		*Other axle boxes.*	
Tin	85	Lead	75
Antimony . . .	10	Antimony	13
Copper	5	Tin	12

These are known as the Nos. 1 and 3 Association of Railway Locomotive Engineer's mixtures.

The metal pockets should be about 3/4 inch deep when the brass is new and slightly undercut to assist in keeping the pad in position. Before white-metalling the surface should be carefully cleaned and tinned so as to make sure of a good metallic contact. No dirt or scum must be allowed to enter when pouring the metal. Figure 2 shows an example of a brass in which the first signs of heating are apparent. It will be noticed that the metal has started to melt away from the edge of the pockets. It is suggested as a possible cause of the heating that as this edge would be the uppermost when the metal was being run in, the scum has risen to it, and has been the cause of a development towards a heated bearing. Another cause for the metal melting away along this edge of the pocket may be that the maximum pressure is exerted on the brass somewhere lower than this edge; that the brass is overheated in this area and the heat developed melts the white metal nearest to it. The bearing area of the brasses for the coupled axle boxes extends over an arc of about 160°, the remainder of the 180° arc being slightly recessed away. The brass should be bedded on over the whole of the 160° arc. These notes are not intended to cover the constructional work of an engine in connection with heated bearings, and such matters as rigidity of the engine framings; careful marking out; erection and correct fitting of the boxes and brasses, are taken as a *sine qua non,* only being mentioned here as being closely associated with the work that pertains to the running department. One more point which should be noticed, however, is the oil groove. The oil is fed in through the crown of the box, but is carried to the bearing surface through this groove and in the case of coupled axle boxes it is the writer's opinion that the groove should approximate very nearly to a circle so as to feed well in towards that part of the bearing where the maximum pressure is exerted. With bogie and tender

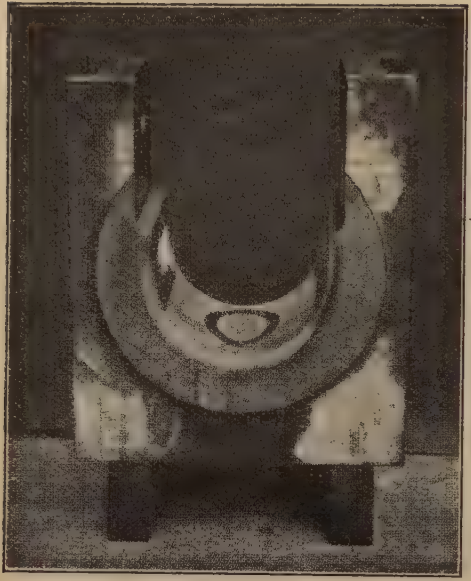


Fig. 1. — Modern driving axle box.
(*See also drawing fig. 5*)

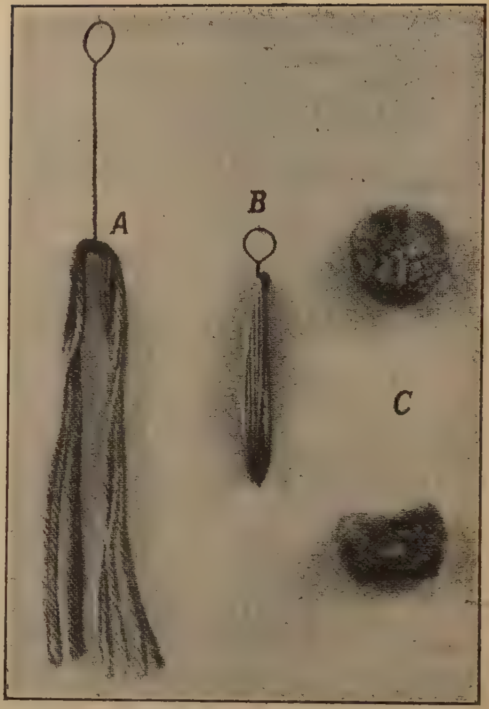


Fig. 3.

A. Tail trimming for coupled axle boxes.
B. Plug trimming for connecting rod big-end joined.
C. Pompon trimmings for ditto (*see fig. 6.*)

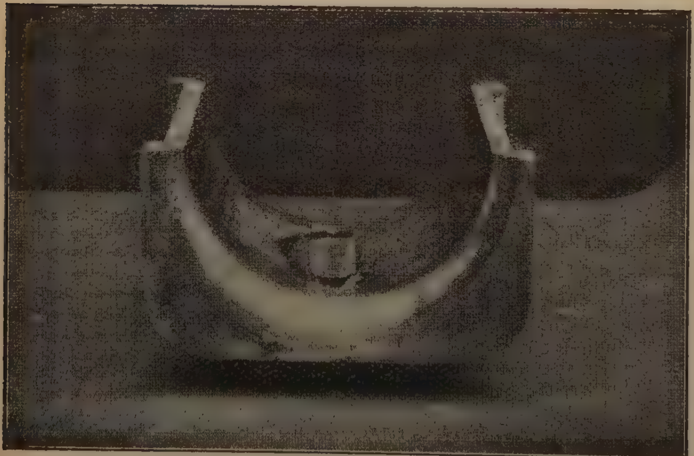


Fig. 2. — Axle brass showing signs of heating.
Note the white metal melted from the edges of the pockets and dragged into the oil groove.

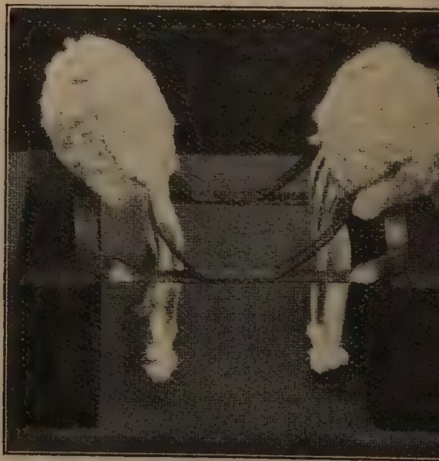


Fig. 4.
Underfeed pad for coupled axle boxes.

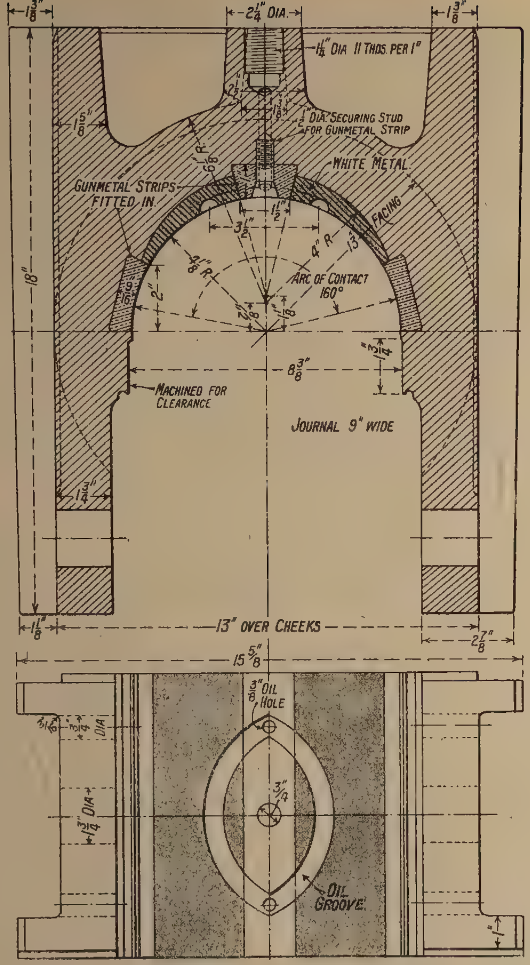


Fig. 5. — Modern axle box of manganese bronze for coupled wheels.

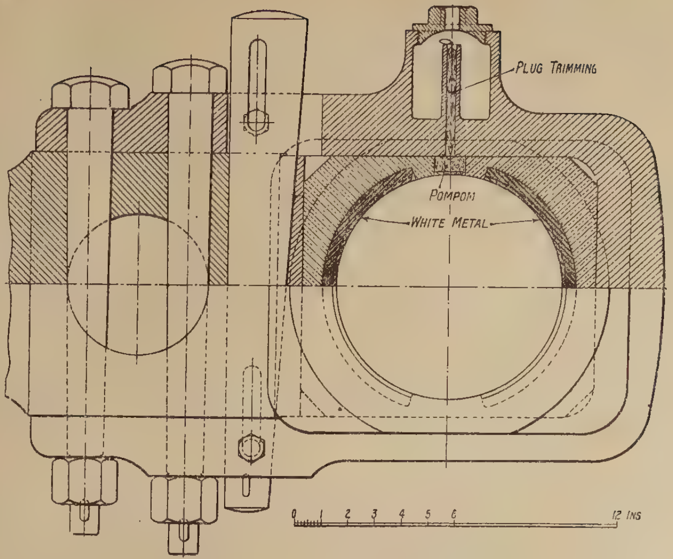


Fig. 6. — Connecting rod big-end.

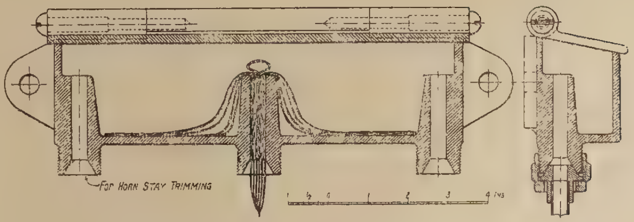


Fig. 7. — Section through oil box, showing trimming in position.

axle boxes fitted with under-feeds it is usual and best for the whole of the bearing area to be white metalled over and for oil pockets to be arranged by applying a 3/4 inch diameter drill for a short depth and at various points over the bearing area.

In figure 4 an illustration is given of a typical underfeed pad for a driving axle box. These pads are essential to satisfactory running. In the case of bogie and tender boxes, they are all that is necessary, the oil being fed directly to them in the underkeep. With the driving and coupled boxes it is the practice to feed the oil to the crown of the journal and for the pad to take up the surplus oil and act as a subsidiary supply. The frame of the pad is of expanded metal and trimmings of pure wool are fitted on to this framing with tails which dip into the axle box underkeep, while the pad is kept in contact with the journal by means of under springs. In the case of some engines cotton waste is packed into the underkeep to serve as a pad, but the wool trimming must be considered much the more satisfactory. It cannot be too strongly emphasised that every driving or coupled axle box should be fitted with pads, as in the case of various classes of engines which have been giving trouble when not so fitted, it has been found that the simple expedient of a slight alteration to the underkeeps so as to make them capable of taking these pads has reduced the trouble enormously.

A word may be added here regarding the journals themselves. In the case of new engines or those whose journals have received attention when an engine has been through the shops it will be found that these are highly finished either with emery and oil, or roller burnishing. When a bearing has been heated, the journal is generally more or less scored or grooved.

If the heating has only been slight the journal may require no special attention, while in the worst cases the wheels require sending to the lathe for the journals to be returned and polished. In intermediate cases it is sufficient to file the journal round with a smooth file and to polish to a finish with emery paper and oil.

A further point which may be mentioned is the question of side clearance of the brasses. This should, of course, be fixed for every class of engine by the designer, but the total side clearances usually specified are 1/8 inch for all coupled boxes of passenger engines, and 1/8 inch for the driving and for the leading and trailing boxes of freight engines 1/4 inch are allowed. For tender boxes the total clearance is usually 1/4 inch.

Method of applying the lubricant

In the case of the larger modern engines it is usual to have mechanical means of lubrication supplied for driving and coupled axleboxes, but a large percentage of the existing stock still uses the oil box from which a worsted « tail » trimming supplies oil through a suitably disposed pipe to the journal. A trimming for use in the driving boxes is illustrated in figure 3 (A), and a sectional sketch of an oil box showing one of the trimmings in position is also shown (fig. 7). This method of lubrication has been in use for a great number of years, but it has the inherent disadvantage that the lower the oil gets in the oil-box the slower the rate of feed. This objection is of most importance in the case of long non-stop runs, when it might even be of advantage if the rate of flow of the oil were to increase as the journey proceeded. This difficulty can be minimised by using oil boxes of comparatively large horizontal cross-sectional area and small depth.

Other methods have been tried, but the present tendency is to adopt mechanical means of lubrication by means of pumps for giving a regular supply of oil throughout the journey. A description of these pumps is unnecessary here, but is should be noted that the drive for them is taken from some part of the engine motion, which gives a movement which shall be as constant as possible for all conditions under which the engine may be working. This drive is so arranged as to be capable of giving a variation in the oil-feed by means of a lever with suitably disposed holes for giving a longer or shorter travel to the pump rams. When the engine is being broken in, the oil supply can be augmented, but normally the supply is adjusted so as to give approximately 2 ounces per axle box per 100 miles. Even with mechanical lubricators, however, careful watch must be kept on the rate at which oil is supplied to the journals as weather conditions may readily affect the flow of oil. The method of supplying the lubricant to the bogie and tender boxes has already been sufficiently described.

Lifting the engine for a «hot box».

There are two methods in use for lifting engines for axle-bearing trouble. One of these consists of the drop-pit method which was described in *The Railway Gazette* for 27 January 1922, and the other, of lifting the engine either by means of sheer legs or a crane. A method in use at some sheds is to use the breakdown crane for this purpose, and when this is used it is generally arranged for economical reasons for at least two engines to be lifted at one firing of the crane, either for hot boxes, « service » repairs or crank axle examinations.

Details will now be given of the actual lifting operation for a bogie passenger engine.

Operations for lifting engines for hot driving axle box with electric sheer-legs.

1. Uncouple engine from tender.
2. Shunt engine on to a sheer-legs pit.
3. Empty boiler.
4. Take front vacuum pipe off.
5. Place packing on top of trailing axle boxes.
6. Take side rods off.
7. Take sand traps and pipes and stays off; empty sandboxes.
8. Uncouple brake rods.
9. Driving brake hangers and stretcher to remove.
10. Bogie centre nut to take off.
11. Cylinder exhaust pipes to remove.
12. Bogie brake steam pipe to uncouple.
13. Back gear eccentric rods to take down.
14. Fore-gear eccentric rods to uncouple at foot.
15. Both connecting rods uncouple and tie up to weigh-bar shaft.
16. Flexible oil pipes to uncouple on driving boxes.
17. Driving stay bolts to remove.
18. Long cylinder exhaust rod to take down.
19. Lifting engine with sheer-legs, taking bogie and driving wheels out, lowering engine down on packing; securing trailing wheels with scotches and replacing horn stays.
20. Replace all of the above when axle box repairs have been carried out.

Repairs to axle box and journal.

1. After trying the brasses over for roll, knock both axle boxes and springs off journals.
2. Try axle box over in the horns for knock.
3. Box, brass and stay to « boil » for cleaning and thence to the white metal shop for metalling as required.
4. Fit brass in axle box.
5. Send box to machine shop for planing on the sides for the horn-plates and boring to journal.
6. While waiting for the machining examine horns and bolts.
7. As already mentioned, it is sometimes necessary to have the journal turned up or filed up if worn or grooved in consequence of the heating.
8. After cutting oil groove, fit box up to the horns and then bed carefully on the journal over the whole of the machined radius.
9. File radius for clearance on boss side of wheel and draw file and scrape for finish.

10. Trammel boxes on the journal to ascertain if in the centre of the horns.

11. Take brasses out of boxes for cleaning and replacing boxes and springs.

In the case of an express passenger engine, when the repairs have been completed the engine should be steamed and worked for a day on yard shunting and for a second day on some local line for a light trip of 30 to 40 miles. It may then be put into traffic, on slow passenger trains to begin with, and generally working up through about ten days to the fastest express trains. With all engines a gruadual breaking in is essential.

Some of the causes of heated axle bearings.

Notwithstanding the heavy loads to which locomotive coupled axle boxes are subjected, it is a fact that they run in the great majority of instances without giving any trouble whatever. This is so much the case that some go so far as to consider that unless any direct mechanical defect can be ascribed as the cause of a hot bearing, then the responsibility lies with the driver for having neglected his duty in supplying the requisite oil to the journal in question. This is undoubtedly the cause in some cases, but we will now consider some of those circumstances which may be the cause, or the partial cause, of trouble developing, and some of these will be found to be outside of the two causes above assigned.

1. Drivers omitting to supply oil; supplying wrong kind of oil or omitting to place trimmings in position.

2. Unsatisfactory fitting of brass or box on journal.

3. Insufficient end play allowed.

4. Axle box too tight in horns or on twist due to horns spreading.

5. Incorrect setting of axle boxes after replacement.

6. Improper distribution of weight — defective spring adjustment.

7. Dirty trimmings — water in oil boxes or under-keeps.

8. Defective oil pipes.

9. Air lock in pipe line with mechanical feed.

10. Grit or sand on journal (leakage from sand boxes).

11. White metal pockets wearing thin and metal breaking away.

12. White metal « scum » poured into pockets during casting.

13. Defective oil pad springs.

14. Roll or knock on bearings.

15. Excessive train loading — actual, and also due to wind.

16. Jar on crossings.

17. Mechanical lubricator defects.

The cost of «lifting», etc., of an engine for a heated bearing.

It may be thought that this is a fairly elementary calculation. As a matter of fact, although as already indicated, there are a great number of operations to be performed, and the cost of these must all be worked out in detail, the calculation is involved with « overhead » charges. Moreover, it is difficult to decide on what is a really typical case, for in one instance a new brass may be required, and the wheels require sending to the journal lathe, while in another both these items may be absent. Other considerations make it inadvisable to quote actual figures, but this is no reason why interested persons should not work out for themselves a specific case appertaining to their own known conditions.

The cost of lifting and replacing with all necessary intermediate operations which is usually performed by a fitter and his mate would form the first item. Having decided on the conditions pertaining to a particular case, the cost of machining and metalling would have to be worked out. If new material is required, the cost must be calculated and credit for scrap material must be allowed for. But to the foregoing must be added a figure, even more difficult to calculate, and this is due to the fact that a valuable piece of mechanism is out of service for some days — the actual period constituting another variable, depending on the lifting appliances and other conditions obtaining. Even when the repairs are completed the engine must be « broken in » on generally less remunerative work than that for which it is designed, and frequently a day is given for « light » running, when enginemen's wages, coal and oil must be taken into account.

The particular kind of box will have to be considered, and in the above it is one of the coupled boxes that the writer has had in mind. For a bogie or tender box the calculation would generally be simpler, and the expense very considerably less. Reverting, however, again to the passenger engine « coupled » boxes, it would be interesting to know the actual number which run hot on the whole of the railways in Great Britain in one year. A conservative figure would be 120 hot bearings for every 250 engines per annum. Calculating on this figure, the number of hot-bearing cases for the coupled axle boxes of passenger engines alone for a railway with 10 000 engines would work out at :

$$\frac{10\,000 \times 120}{250} = 4\,800 \text{ per annum,}$$

and the cost of dealing with these, computed at a figure arrived at as explained above, would reveal an expenditure which would show that some concentration with a view to at least partial elimination is well worth while on the part of all concerned.

Question of supplying higher grade oil.

In the case of engines which give repeated trouble with heated axle bearings, one obvious method of attempting to eradicate the trouble is to supply a higher grade of oil. An engine with two pairs of coupled wheels may be assumed to use 2 lb. of oil on an average for each set of men using it per working day. The engine may be used by more than one set of men on some days, so that as a fair figure, 3 lb. of oil may be taken as being used per day. This would work out at two gallons per week. Taking an actual case where such an alteration was made with satisfactory results, a higher grade of oil was used with an increase in cost per gallon. By taking the increased cost of oil per week and dividing it into the cost of lifting an engine for a heated bearing we arrive at the number of weeks necessary to disburse this sum of money in the higher grade oil. In a certain case this worked out at three hundred weeks, and as the heated bearings on the engines in question had been comparatively frequent, and the introduction of the new oil most effectively reduced them, it will be seen that its introduction seems to have been justified.

A note on connecting rod big-end journals.

On some railways heated big-end journals give a good deal of trouble, and it is the writer's opinion that this condition of affairs, with a certain amount of necessary attention, can at least, be improved. In the case of one depot with 200 engines, there have not been half-a-dozen heated big-ends for the last six years. This is due to the fact that the original design of the big-ends (fig. 6) is sound, that one specially trained and reliable fitter carries out at regular intervals (5 000 to 6 000 miles) practically all the big-end examinations, and to the method of lubrication of the journal. It will be seen from the drawing that the oil is fed to the big-ends by a plug trimming (B in fig. 3) fitting in a 5/16-inch diameter hole. This trimming is made with 28 strands of wool and feeds down to a woollen pompon, made similarly to a child's woollen ball (C in fig. 3) ; sponge is unsatisfactory for this purpose; the pompom must be made of wool. To receive it, it may be necessary to open out the brasses as shown in the drawing, and it can with confidence be asserted that the adoption of this simple contrivance will lessen any trouble that may be experienced with these journals.

The brasses must be so fitted that they bed well to the journals, but are at the same time quite free and no « draw » whatever must be left. With these points watched, assuming the original design to be sound, little trouble should be experienced with big-end journals.

Summary and conclusions.

It may be thought that more has been said in these notes of the facts relative to the heating of the boxes, rather than to any effective way of eradicating the defects, but it will be appreciated that with designs so well established as they are in locomotives, far-reaching changes are almost out of the question and the problem is to make the very best conditions to obtain with the existing designs. The cure can only be effected if those responsible for performing it have full and complete knowledge of the facts and to this end every hot bearing should be reported to headquarters. It is the duty of the officer responsible for this work to classify the reports; to see which engines are giving most trouble in this respect and then by investigation to ascertain the best means of preventing it.

In certain cases drivers are to blame and they must be dealt with; and even in doubtful cases it is advisable to see the drivers, to question them as to whether or not their duties were satisfactorily performed, and if they say they were, to point out to them that no definite cause can be assigned to the case, but that their care in the matter is essential, so as to avoid expense to the railway company and inconvenience to the travelling public.

A regular examination period for crank axles with all coupled boxes is desirable, and for passenger engines such a period should be based on mileage worked, at 20 000—24 000 miles. At this time the boxes receive attention and roll in the boxes and knocking in the horns eradicated. Drop pits may somewhat reduce the time an engine is out of service for hot bearing trouble but the effective way seems to be to concentrate on the trouble itself.

Ready accessibility of all oil boxes should be insisted upon, and all coupled boxes should be provided with white metal pads and the underkeeps should be fitted with woollen oil pads. It is also essential that all oil pads should be thoroughly soaked in oil before fitting to the axleboxes.

In some instances it may be advisable to introduce a higher grade of oil or increase the feed of the oil. Careful fitting is essential. If possible, one man should be kept on the big-end journal work and every axle box or brass should be passed before replacing by the foreman in charge. Mechanical lubrication for coupled axle boxes is recommended for all new stock designed for main-line working.

CURRENT PRACTICE.

Under this heading we intend, to the largest possible extent, to keep our readers regularly and quickly informed concerning the more interesting new devices and other improvements introduced in permanent way practice, in rolling stock, and in operating equipment, as well as in the organisation of the different railway services in countries to which the activities of the International Railway Congress Association extend.

The Executive Committee.

[621 .132.5 (.73) & 625 .245 (.73)]

New motive power and rolling stock on the Reading Company.

Figs. 1 to 4, p. 206 and 207.

1. — 2-10-2 Santa Fe K-Isa engines.

Figure 1 is the photograph of the K-Isa engines, two of which, Nos. 3000 and 3001, have been completed. Locomotive 3001 recently completed at the Reading shops is the second of the 2-10-2 Santa Fe type to be placed in service by this Company.

In order to provide for higher train speeds, old compound *Mallet* engine 1800 was taken into the Reading locomotive shop, dismantled, and changes and repairs made to boiler and rebuilt into a Santa Fe engine No. 3001.

Engine 3001 is one of the largest and heaviest two-cylinder simple Santa Fe engines built up to this time, having a total weight in working order of 438 200 pounds or 219 tons. The driving wheels were enlarged to 61 1/2 inches in diameter and in spite of the large size and heavy weight, movement of the engine is very free and easy even when passing curves, operating easily on 16° curves. The main frames and crossheads are made of special vanadium alloy cast steel, giving an increased strength of approximately 33 %.

The frames are connected and braced with five heavy box section cast steel crossties bolted to each frame. Two of these crossties serve as supports for the brake cylinders, four of which are required, two 12-inch to brake the two front pair of drivers and two 14-inch braking on the main and two rear pair of drivers.

The boiler, except for a few changes, is the same as used on engine 1801, and has a diameter of 102 inches over the largest course. The firebox is 9 feet wide by 12 feet long, giving 108 square feet grate area.

The main rods are made with the solid back end having floating brass bushing, the only adjustment required

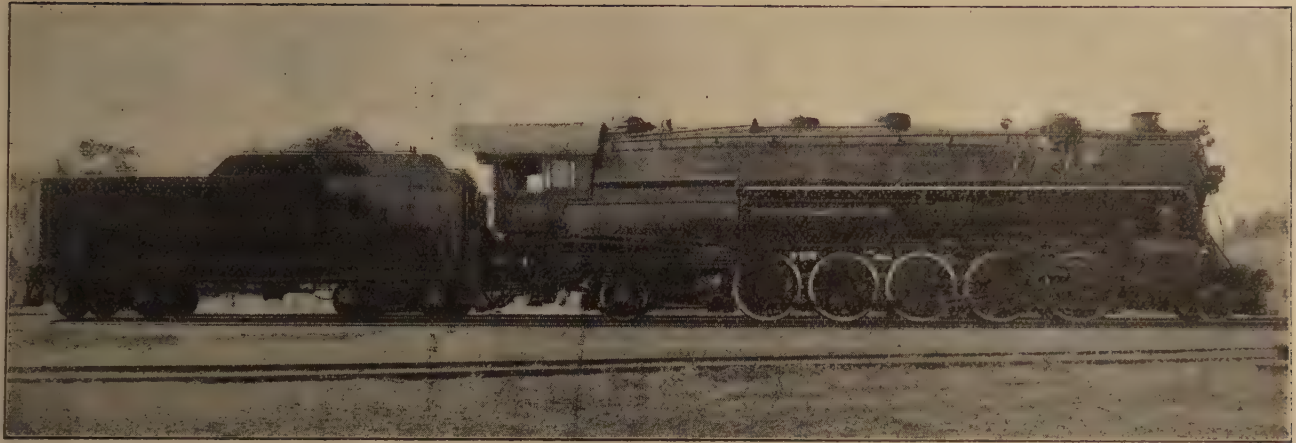


Fig. 1.

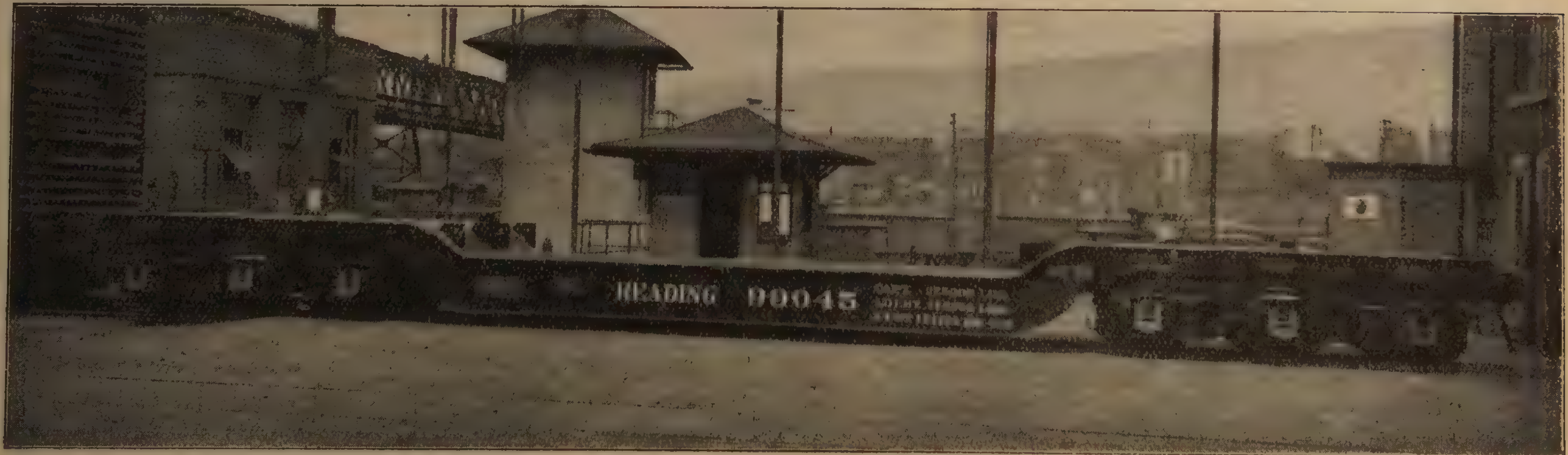


Fig. 4.

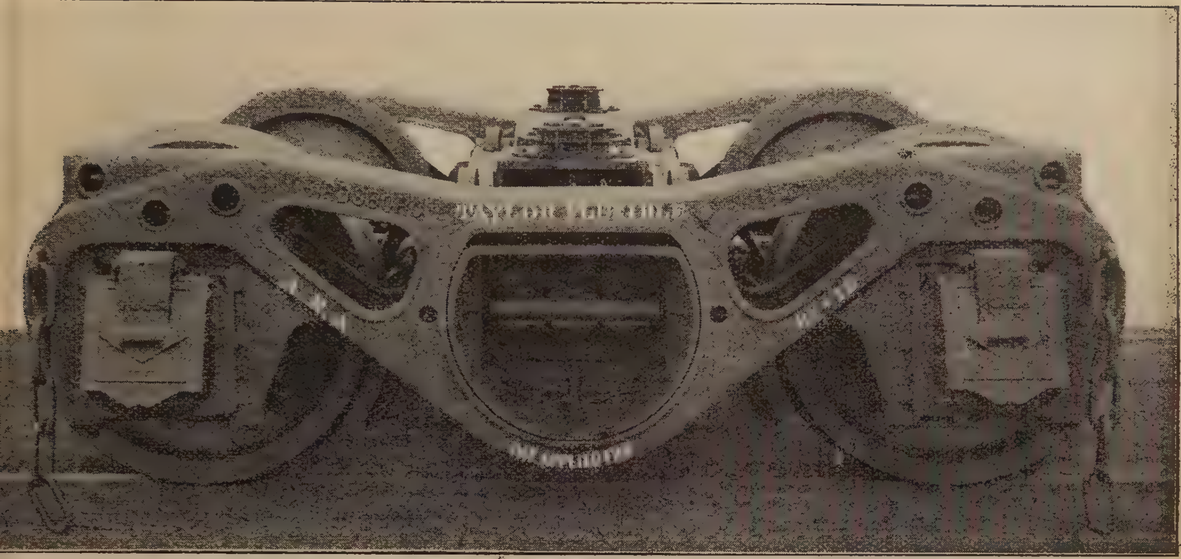


Fig. 2.

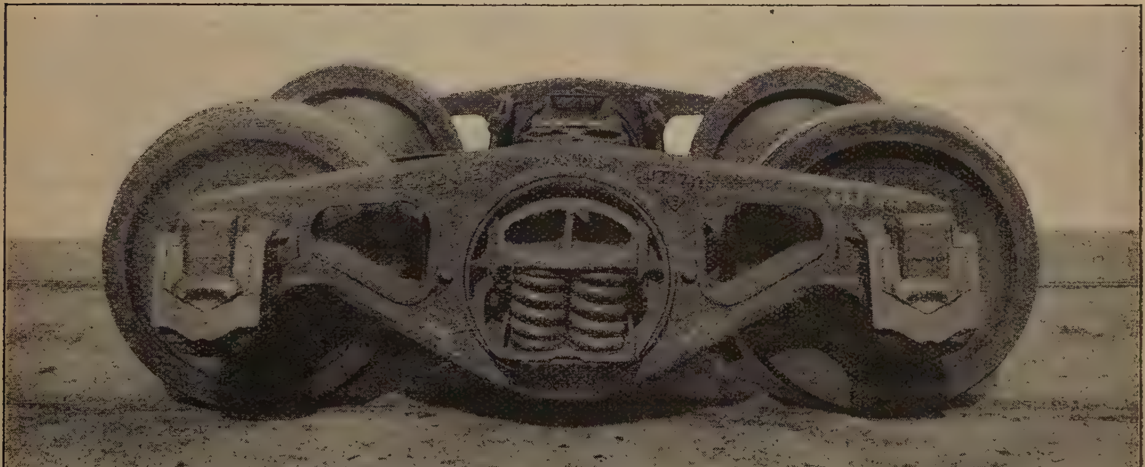


Fig. 3.

being a new bushing when the limit of wear is reached.

The arrangement of the cab is neat and roomy, space being provided to seat three men comfortably on the left side.

The engine is provided with a new and larger tender, having a capacity of 12 000 gallons of water and 18 tons of coal, the tender frame being of the Commonwealth type of cast steel, made in one piece and fitted with 4-wheel flexible side frame cast steel trucks having 6 1/2 × 12-inch journals.

The equipment on the engine consists of two Westinghouse cross-compound pumps, Sellers 14 1/2 exhaust steam injector, Ashcroft cut-off control gauge, Dupont Simplex type B Stoker, Franklin Butterfly firedoor, Hancock air cylinder cocks and B & S automatic drifting valve.

Figure 2 is the photograph of the Taylor flexible tender trucks with which these engines are equipped.

2. — New freight trucks without equalizers.

Figure 2 is the photo of the 6×11-inch freight trucks. The installation of these tender trucks saved 1 700 lb. per truck or 3 400 lb. per tender and makes a very flexible, easy riding truck in fast freight and passenger service. Due to the frames performing the function of the equalizers, the equalizers are omitted, thus making a saving in weight even though the castings are slightly heavier than the ordinary parts. Due largely to the substitution of a round hole for the transom and bolster in place of the usual square opening the strength has been materially increased. This has been borne out by actual tests to destruction. Another advantage is the elimination of eccentric loading of the spring nests in case of bad track or derailment. The ordinary bolster and side frame, in case of derailment, will change their spring bearing surfaces from the usual parallel position to an angular one which naturally results in throwing an excess of load on certain springs of the nest. This is obviated in the case of the Taylor truck, as an eccentric loading results in the ring or transom, as the case may be, rotating and maintaining the upper and lower surfaces of the spring next parallel. The freight truck is now in service on over a thousand cars of 6 × 11-inch axle capacity.

3. — Heavy-duty depressed-center flat cars.

Figure 4 is the photograph of five heavy-duty depressed-center flat *cars* recently placed in service by the Reading Company. The entire body portion of these cars is a single steel casting wheighing approximately 57 000 lb. each. The trucks of these cars are of the six-wheel equalized type with clasp brakes, the truck frame being a single steel casting. Both truck frame and body were manufactured by the Commonwealth Steel Company.

[628 .232 (.44)]

Bogie saloon-coaches,

belonging to the Paris, Lyons and Mediterranean Railway,

specially fitted out for the conveyance of invalids.

Fig. 1, p. 210.

In building and equipping its special invalid carriages, the Paris, Lyons & Mediterranean Railway Company has done everything possible to ensure the maximum comfort under the most hygienic conditions.

The invalid compartment measures 4.5 m. × 2 m. (14 ft. 9 in. × 6 ft. 6 3/4 in.) giving an air space of 22 cubic metres (777 cubic feet) with direct access to a dressing room, also used as a service room.

The fittings consist of a low bedstead in polished brass which can be moved into any position the invalid wishes. The head of the bed is usually placed against the partition between the bedroom and the dressing room so as to give access to three sides of the bed. The spring mattress has very flexible flat springs and can be washed down.

At the opposite end of the compartment a second low polished brass bedstead is arranged transversely, and in the daytime can be converted into a couch. The invalid in consequence can have near him the nurse in whose care he is, during the day as during the night.

In the invalid compartment all projections capable of holding dust have been proscribed. All corners have been rounded, the walls, ceilings and floor are formed of washable material which can de disinfected after each journey. The carpets covering the linoleum and the curtains over the windows can be washed.

The whole section and its equipment guarantee perfect cleanliness.

On each of the sides of the compartment a large double door is provided to make it easier to carry in the invalid. The door into the corridor can be closed up in cases of contagious diseases so as to ensure complete isolation.

Large windows give ample light by day : they can be covered over by curtains in washable material or be completely closed by shutters.

The electric light fittings are carried in the roof and give at will either a very strong or a very soft light. They can all be switched off completely. In addition, the invalid has a reading lamp at the head of his bed so that if he so wish he need not use the roof light.

Various plug-in points with flexible leads provide every facility required during the journey when making examinations, dressings, etc.

Protection against temperature variation is assured by lining out the roof and body sides with cork insulating material and by double glasses in the outside lights. When these two lights are closed the air between them hinders loss of heat or penetration of cold. During winter condensation on the glass is also avoided in this way.

Plain shutters can also be raised up into position on the inside of the lights, thereby adding to the protection against cold and ensuring the complete exclusion of light from outside.

Large steam radiators of a perfected design with smooth sides are placed along the sides of the compartment below the lights and enable a high temperature to be obtained even in the coldest weather, whilst at the same time the temperature can be regulated as desired. The air passing through the heaters is drawn in from outside and passed through filters consisting of layers of cotton so that the air supply is completely freed from dust.

If it is desired to raise the temperature without introducing heated air, an electric radiator or an electrically heated blanket is available.

The large radiators can be used in summer to lower the temperature : iced water can be passed through them instead of steam.

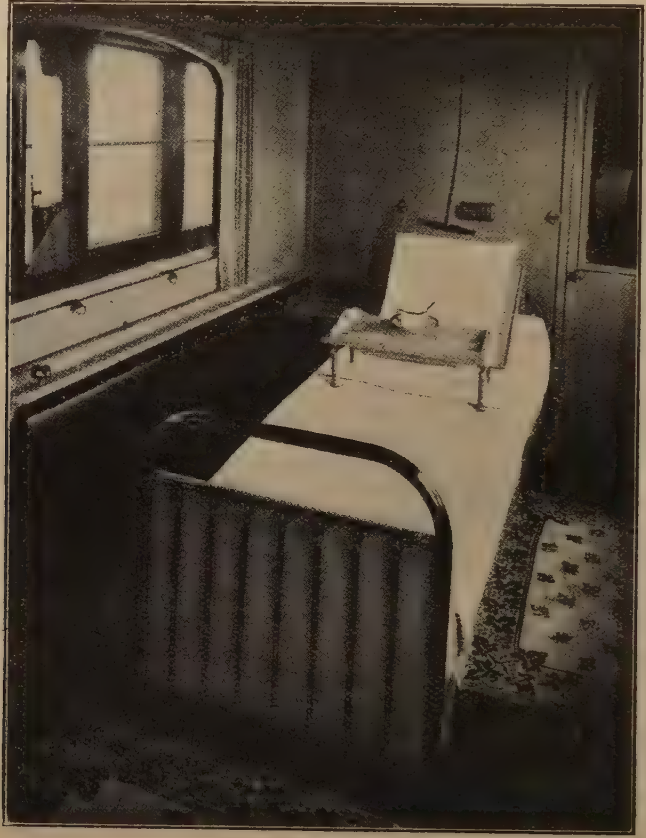


Fig. 1.

The accessory fittings provided enable the invalid to raise himself in bed, to set the bed into a sloping position at different angles, to take his meals in bed, to read from a reading desk, etc.

The cork insulation, in addition to preventing changes of temperature, has also been arranged to reduce noise and damp out vibration.

The dressing room attached to the compartment contains a flushing hopper, an adjustable bidet, and a wash basin with hot and cold water.

In the event of contagious illness, the

hopper empties into a hermetically closed container in order to prevent infectious matter being discharged on to the frame of the vehicle or the track. This container is thoroughly disinfected each journey.

The service part of the dressing room contains a fountain with hot and cold water, a refrigerator, an electric bain-marie, electric heaters for preparing hot foods and drinks, a medicine and dressings cupboard, a cupboard for linen, vessels, etc., a hermetically closed box for soiled linen, etc.

A special cupboard in the carriage contains a Rouvillois stretcher for carrying seriously wounded cases or invalids unable to move by themselves. By means of this stretcher, the invalid can be taken from his bed, transferred directly to the bed in the carriage without other change, and on arrival again moved in the same way to his bed at home.

MISCELLANEOUS INFORMATION

[625 .236 (.42)]

1. — Carriage washing plant at York, London and North Eastern Railway.

Figs. 1 and 2, p. 213.

The Railway Gazette.)

The maintenance of coaching stock in a clean and attractive condition is a very important factor in the securing and retention of passenger traffic to the railways, but the cleaning operations necessary are extremely expensive in labour cost. With a view to improving the cleaning of the carriages and at the same time reducing to some extent the heavy expenditure, the London & North Eastern Railway has installed at Clifton Carriage Sidings, York, apparatus for the mechanical washing of the outsides of carriages. The apparatus takes the form of eight vertical mops, or wipers, placed in two rows, four at each side of the track, enclosed in a brick building built over the shunting neck from which access to all the sidings in the carriage sheds is possible.

The mops consist of vertical iron shafts to which are affixed thousands of strips of cloth each measuring 4 inches in width and 25 1/2 inches in length. The mops are rotated in opposite directions by means of electric motors at a speed of 50 revolutions per minute, the motors with the necessary gearing being placed on the framework above the vertical shafts. The cloth strips are saturated with water directed upon them from above, and suitable drains are provided in the concrete floor to carry away the water.

During the operation of washing the coaches travel at the rate of about one and a half miles per hour, and on entering the machine, but before reaching the mops, they are sprayed with water from jets at both sides. As the mops rotate, the saturated cloth strips are flung out at right angles to the shafts and whirl vigorously against the sides of the vehicle, effectively removing all dirt. The length of the strips provides for a large cleaning surface being applied to the vehicles, and their flexibility permits of automatic adjustment to compensate for variations in the widths of different coaches and for irregularities in the width of each coach, such as inset doors and window frames, and projecting door and commode handles. As the vehicles are leaving the machine a final spray of clean water completes the washing and ensures that dirt loosened by the apparatus is rinsed off before the coaches are allowed to dry.

After the vehicles have passed through the machine no further external work upon them is necessary. The windows can safely be left to dry without being wash-leathered, as the final clean water spray prevents any streaks to dry without being wash-leathered, as the the revolving cloth strips polishes brass door and commode handles.

A bogie vehicle can be washed by means of the apparatus in from one and a half minutes, and the results are better than those obtained with ordinary hand washing by reason of the increased force of the power-driven mops. At

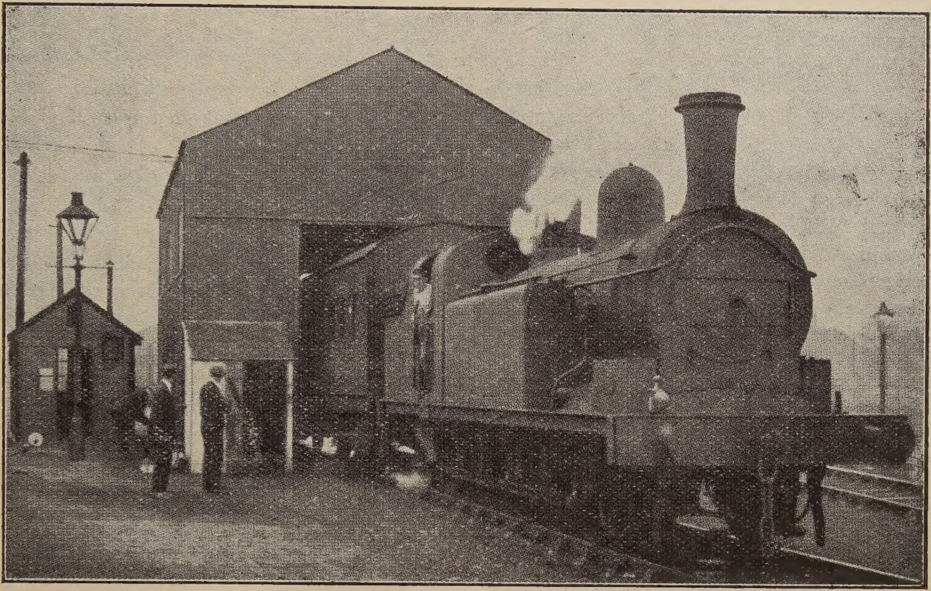


Fig. 1. — Train passing through washing plant.

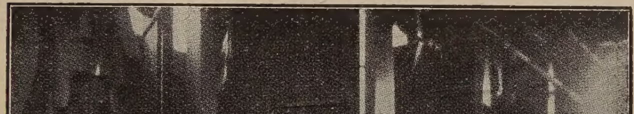


Fig. 2. — Attending to the rotary washers.

the same time, the action of the apparatus is not so violent as to damage the paint and varnish of the vehicles.

At the present time it is found that ordinary washing by hand is not sufficient alone to keep the vehicles in a clean condition, and periodical special cleaning of vehicles by means of acid preparations is necessary in order to remove dirt and grease, which, after the ordinary washing with water by hand labour, is left adhering to the coaches. The final operation of the special cleaning by means of acid preparations is for the vehicles to be washed down with water, and the machines will be utilised for the performance of this work. It is, moreover, anticipated that the very efficient removal of dirt by means of mechanical water washing will result in the special cleaning referred to being required only at longer intervals than has hitherto been the case.

One satisfactory feature of the machine is that as the plant is divided into two sections, each of which is driven by a 15-H. P. motor, it will be possible for one half of the plant to be laid off for repairs while the other is continuing the performance of its work. In these circumstaces the vehicles will require to be shunted through at about half the speed previously mentioned.